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CONCRETE
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Volume LIII, No. 12. LONDON, DECEMBER, 1958.

EDITORIAL NOTES

Unnecessary Accuracy.

THE uncertainties inherent in the design and construction of reinforced concrete are so many that for most structures little or no attempt is made to predict the magnitude or even the nature of all the stresses. Superficial and much simplified calculations are generally made which are known to ignore many factors and the results compared with the recommendations in a code of practice which are known to allow for such factors; that is the secondary effects may be ignored because they are included in the factor of safety. The justification for this procedure is that for many years it has resulted in satisfactory structures, and it is foolish not to take advantage of experience.

For example, in designing an ell-beam the effective breadth of the flange is estimated empirically, and the effective depths at midspan and at the supports and the amount of reinforcement required are obtained by simple calculations based on the resistance to bending and shearing of an assumed ideal material which bears little relation to concrete. The breadth of the rib is determined by calculating a nominal shearing stress, or by the space required to accommodate the reinforcement. No calculations are made of the stresses due to creep, shrinkage, torsion caused by unbalanced loading, or normal changes of temperature, because the working stresses recommended by codes and regulations are small enough to allow such secondary stresses to be present without danger. Because of these uncertainties, it is also rarely necessary to determine precisely the loads and bending moments acting on a structure. All except the simplest of concrete structures are statically indeterminate, generally to a very high degree. The exact distribution of the load imposed on a beam by a slab which is monolithic with it, or of the bending moments acting on a continuous beam with elastic supports of finite width, are difficult, and sometimes impossible, to calculate. Approximations are therefore used; it is assumed that the load imposed by the slab is uniformly distributed, and that the beam has rigid supports of infinitesimal width. The results of the calculations are approximate and conservative, and as they are generally based on nominal uniform loads, and are used in perfunctory calculations to determine the dimensions of the members, greater accuracy is useless.

Most engineers are fully aware of the futility of basing " exact " calculations

on inexact data, but the belief that approximate methods are unsafe or otherwise unsatisfactory is still sufficiently widespread to be the cause of much wasted time and effort. In a recent design for continuous beams of equal spans calculations were made to determine the bending moments corresponding to all possible combinations of live loads on adjacent and alternate spans. The greatest bending moments at the supports were then reduced by fifteen per cent., and the bending moments at midspan were adjusted accordingly. As would be expected, the results of this lengthy exercise in arithmetic were almost identical with those given by the approximate coefficients which are generally used. The confusion of arithmetical accuracy with good design generally results only in a waste of the designer's time, but more serious consequences may occur. For example, a small retaining wall was built some years ago at the side of a garage; the wall overturned and was re-built to the same design, which the designer had carefully checked. The re-built wall also overturned, the calculations were submitted to another engineer to check, and it was found that although the arithmetic was impeccable no allowance had been made for the weight of loaded lorries parked immediately behind the wall. It is evident that the designer was so absorbed in arithmetic that he lost sight of a basic requirement.

There can be few designers who have never found that attempts at apparent arithmetical precision have tended to cloud their engineering judgment. Most employers discourage unnecessary precision, which wastes time and money and does them no credit when such work is seen by others. Indeed, the shortage of skilled designers of which so much is heard could be more accurately described as a surplus of relatively unskilled ones; if all unnecessary drawing-office work were avoided it is possible that calculations and drawings would be made at the same speed as before the war and there would no longer be an unsatisfied demand for designers and detailers.

There are many less obvious examples of waste than those mentioned in the foregoing, and which are of much more frequent occurrence. The arithmetical processes involved in such methods of design as moment distribution are often pursued to an unnecessary degree of accuracy, simply because the mechanical nature of the work leads to a temporary suspension of thought. Lightly-loaded slabs may be analysed and the reinforcement calculated, when a little thought would show that the minimum reinforcement permitted by the code of practice would be more than sufficient. Also, restraint moments may be calculated by more or less complex methods for structures whose unimportance would justify the use of estimated values. Occasionally such labour is performed in the name of economy, but the number of designers who compare the savings of material achieved in this way with the cost of the time taken to make them is surprisingly small. Greater accuracy in necessary detailing would save a great detail of expense, particularly when mistakes are discovered at a late stage.

The increased cost of drawing-office work is frequently mentioned by older engineers, some of whom claim that the output per man to-day is very much less than what it was twenty years ago. The avoidance of unnecessary arithmetic is an obvious way of making up for the fewer hours now spent at the drawing board as a result of the shorter working week and the longer holidays enjoyed by most designers. As in industry, higher costs in a drawing-office can often be countered by labour-saving devices.

Shrinkage Stresses in Composite Floors.

By R. J. BARTLETT.

In beams and slabs of composite construction stresses develop due to the differential shrinkage of the components. The topping is generally cast over precast beams and the two eventually act together structurally. The shrinkage of the topping will be partly resisted by the beams and stresses will therefore be present in the combined section.

Consider the construction shown in *Fig.* 1. If X were not bonded to Y it would shrink unhindered and no stress would occur. Let s represent the shortening per unit length. If this shrinkage were elastic a constant stress equal to s

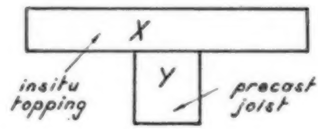


Fig. 1.

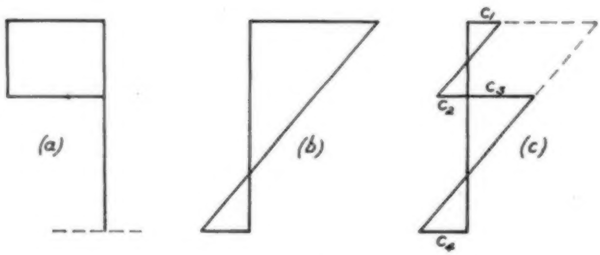


Fig. 2.

times Young's modulus E would be present in X, as shown in the stress diagram in *Fig.* 2(a). The total force F would be $s \times E$ times the area of X. This force acts eccentrically on the composite section producing a distribution of stress as in *Fig.* 2(b). Since, however, force F is not present, the stress due to it acting on X alone must be allowed for in *Fig.* 2(b), producing the distribution shown in *Fig.* 2(c). The final stresses at top and bottom of the composite member are respectively

$$c_1 = \frac{F}{A_x + A_y} + \frac{Fe}{Z_t} - \frac{F}{A_x} \quad . \quad . \quad . \quad . \quad (1)$$

$$c_4 = \frac{F}{A_x + A_y} - \frac{Fe}{Z_b} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

in which A_x and A_y are the areas of X and Y, Z_t and Z_b are the section moduli of the combined areas X and Y, and e is the distance between the centre of area of X and the centre of area of X and Y combined. Since the concretes in X and Y are of different composition and maturity it is advisable to use a modular ratio of, say, 1·5 or 2 in calculating the properties of the combined section. The

stresses c_2 in X and c_3 in Y at their surfaces of contact may be obtained by proportion from *Fig.* 2(c).

The moment Fe is constant throughout the whole of the beam and therefore there are no shearing stresses. At each end, however, the stresses due to bending are zero and increase to a maximum over a short distance. This distance cannot be determined precisely, but from the principle of St. Venant it may be considered to be equal to the depth d of the joist.

From *Fig.* 2(c) it is seen that, in order to maintain the stresses in X, a force P in the plane of contact and a moment M perpendicular to this plane are necessary [*Fig.* 3(a)], and these are supplied by Y at each end of the beam. Conversely, P and M may be found by considering the stresses in Y as being maintained at each end of the beam by X as in *Fig.* 3(b). Thus P and M are calculated from

$$c_1 = \frac{Pe_1 - M}{Z_x} - \frac{P}{A_x} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$$c_2 = \frac{Pe_1 - M}{Z_x} + \frac{P}{A_x} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

in which e_1 is the eccentricity of P, and Z_x is the section modulus of X. Similarly,

$$c_3 = \frac{Pe_2 + M}{Z_{yt}} + \frac{P}{A_y} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

$$c_4 = \frac{Pe_2 + M}{Z_{yb}} - \frac{P}{A_y} \quad . \quad . \quad . \quad . \quad . \quad (6)$$

in which e_2 is the eccentricity of P, and Z_{yt} and Z_{yb} are the section moduli of Y. The solution of (5) and (6) is not essential but is a check on the solution of (3) and (4).

The force P is a horizontal shearing force in the plane of contact of X and Y

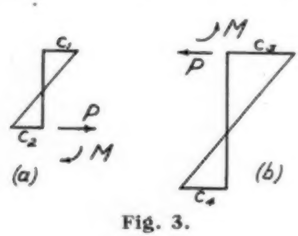


Fig. 3.

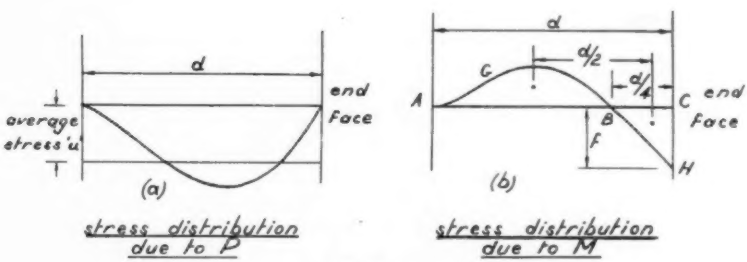


Fig. 4.

at each end of the beam over an area d times the breadth b of the plane of contact.

If F is likened to a prestressing force imposed on the combined section, the distribution of the bond and the bursting stresses due to P and M respectively would be approximately as shown in *Fig.* 4(a) and (b) ("Prestressed Concrete." By Y. Guyon. Pages 128/9).

Thus the average bond stress

$$u = \frac{P}{bd} \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

The maximum bond stress is probably 50 per cent. higher than the average since the distribution over the distance d is approximately parabolic.

The maximum tensile stress f tending to separate the beam from the topping, which would occur at the end face of the beam, may be calculated as follows from the approximate dimensions in *Fig.* 4(b). Area BHC representing the tensile stress must equal the area AGB representing compression. Also BHC is very nearly a triangle since line BH is almost straight. The distance between the centres of areas of BHC and AGB is approximately half the depth. Therefore

$$M = \text{area BHC} \times b \times KL = \tfrac{1}{2} f \frac{d}{4} \times b \times \frac{d}{2} = \frac{fbd^2}{16}.$$

Thus $$f = \frac{16M}{bd^2} \text{ (approximately)} \quad . \quad . \quad . \quad . \quad (8)$$

The stresses due to shrinkage throughout the beam can be high in some cases, but will be substantially reduced by creep in both the beam and the topping.

It is assumed in the foregoing that both concretes are capable of resisting the tensile stresses indicated in *Fig.* 2(c). The introduction of stirrups at each end to assist in resisting P and M would be unavailing if cracking occurred in the topping, since it would divide the beam into two so far as shrinkage stresses are concerned because these stresses are independent of the length of the beam.

Example.

A topping 2 in. thick is cast over and bonded to precast beams at 20-in. centres (*Fig.* 5), Young's modulus is 2×10^6 per square inch for the topping and 4×10^6 per square inch for the beams, and the shrinkage per unit length of the topping is 2×10^{-4}. Calculate the stresses, due to shrinkage of the topping, in the cross section and the bond and bursting stresses at each end of the composite beam. $A_x = 20 \times 2 = 40$ sq. in. $A_y = (6 \times 2 + 2 \times 2 \times 2) \times$ modular ratio $= 40$ sq. in. The distance of the centre of gravity of the section from the bottom is

$$\frac{40 \times 7 + 2 \times 12 \times 3 + 2 \times 8 \times 1}{40 + 40} = 4{\cdot}6 \text{ in.}$$

The moment of inertia of the section is

$$\frac{20 \times 2^3}{12} + 40 \times 2{\cdot}4^2 + \frac{2 \times 2 \times 6^3}{12} + 2 \times 12 \times 1{\cdot}6^2$$

$$+ \frac{2 \times 4 \times 2^3}{12} + 2 \times 8 \times 3{\cdot}6^2 = 589{\cdot}87 \text{ in.}^4$$

$$Z_t = \frac{589{\cdot}87}{3{\cdot}4} = 173{\cdot}5 \text{ in.}^3 \qquad Z_b = \frac{589{\cdot}87}{4{\cdot}6} = 128{\cdot}0 \text{ in.}^3$$

$$F = 2 \times 10^{-4} \times 2 \times 10^6 \times 2 \times 20 = 16{,}000 \text{ lb.}$$

$$Fe = 16{,}000 \times (3{\cdot}4 - 1) = 38{,}400 \text{ in.-lb.}$$

Using equations (1) and (2),

$$c_1 = \frac{16{,}000}{80} + \frac{38{,}400}{173{\cdot}5} - \frac{16{,}000}{40} = 21{\cdot}5 \text{ lb. per square inch.}$$

$$c_4 = \frac{16{,}000}{80} - \frac{38{,}400}{128{\cdot}0} = -100 \text{ lb. per square inch.}$$

(c_4, being negative, is a tensile stress.)

By drawing the stress diagrams as shown in *Fig.* 6 similar to those in *Fig.* 2, c_2 and c_3 may be calculated from the geometry of similar triangles. Thus $c_3 = (21{\cdot}5 + 400 + 100)\,\dfrac{6}{8} - 100 = 291$ lb. per square inch and $c_2 = 291 - 400 = -109$ lb. per square inch.

The true stresses in the beams are c_3 and c_4 each multiplied by the modular ratio 2; that is

Stress at top $= c_3 \times m = 291 \times 2 = 582$ lb. per square inch compression.
Stress at bottom $= c_4 \times m = 100 \times 2 = 200$ lb. per square inch tension.

To calculate P and M, $A_x = 40$ sq. in., $Z_x = 20 \times \dfrac{2^2}{6} = \dfrac{40}{3}$, and $e_1 = 1$ in.

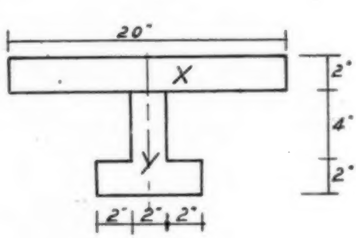


Fig. 5.

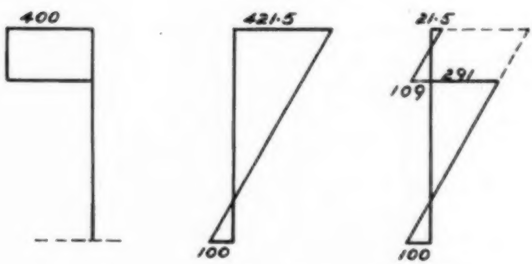


Fig. 6.

Therefore, using equations (3) and (4),

$$21{\cdot}5 = \frac{P \times 1 - M}{\frac{40}{3}} - \frac{P}{40}. \qquad 109 = \frac{P \times 1 - M}{\frac{40}{3}} + \frac{P}{40}$$

therefore $P = 1750$ lb. and $M = 880$ in.-lb.

By calculating Z_{yt}, Z_{yb}, and e_2, and then solving (5) and (6), substantially similar values of P and M are obtained. Then, from (7) and (8), the average bond stress between the topping and the beam $u = \frac{1750}{2 \times 6} = 146$ lb. per square inch.

and the approximate bursting stress $f = \frac{16 \times 880}{2 \times 6^2} = 196$ lb. per square inch.

Book Reviews.

"The Building of T.V.A." By John H. Kyle. (The Louisiana State University Press, Baton Rouge, U.S.A. Price $7.50.)

No attempt is made at giving technical details. The volume comprises mainly photographs of the dams, bridges, power stations built by the Tennessee Valley Authority since it was formed twenty-five years ago, with brief notes on each structure.

"Züblin-bau." (Stuttgart: J. G. Cotta'sche Buchhandlung Nachf. No price stated.)

THE Swiss family of Züblin were among the pioneers of reinforced concrete in Europe. In the year 1894 Eduard Züblin built a reinforced concrete loggia in Naples. In 1895 he became the licensee for the Hennebique system in Italy, where he built reinforced concrete factories and a large tank in the years 1896 and 1897, and in 1898 transferred his activities to Strasbourg. The sixtieth anniversary of the starting of the Züblin business is marked by the issue of this fascinating book, written by Wolf v. Niebelschütz, translated into masterly English by A. Laubenthal, and beautifully printed and bound in Germany. The earlier chapters deal with the development of reinforced concrete in the second half of last century, and the remainder is used to illustrate some of the structures built by Ed. Züblin A.G. in many countries of the world. The book is one of the best sources of information on the early development of reinforced concrete, and the illustrations, although depicting the work of one firm only, present a remarkable record of the variety of reinforced and prestressed concrete structures built in this century. The size and daring of some of the structures built fifty or more years ago will surprise many who are familiar only with more recent concrete work.

"Composite Construction in Steel and Concrete." By I. M. Viest, R. S. Fountain, and R. C. Singleton. (London: McGraw-Hill Publishing Co., Ltd. Price £2 18s.)

METHODS of designing members of structural steel sections encased in concrete, together with formulæ, tables, and graphs, are given in accordance with the recommendations of the American Association of State Highway Officials. The book comprises 170 pages, and is mostly concerned with bridges.

"Bemessungsverfahren." By Benno and Helmut Löser. (Berlin: Wilhelm Ernst & Sohn. 1958. Price 22 D.M.)

THE 16th edition of this handbook on the design of reinforced concrete is in accordance with the latest codes of practice of the German-speaking countries.

In the section on bending of rectangular sections, tables are included for the stresses of 42,670 lb. and 49,780 lb. per square inch permitted for Austrian high-grade IV steel and Torstahl. The properties of special high-grade and cold-worked steels are discussed, and notes are given on Torstahl 40, 60 and 80 and the Czechoslovakian Roxor steel. There is also a section on glass-concrete floors and roofs.

Professor Brendel describes a method

which is claimed to give more accurate results than the usual trial-and-error calculations in the preliminary design of prestressed beams of any shape.

The recommendations of the codes are explained, including the ultimate-load method of design, and numerical examples are given. Many of the formulæ and tables are not directly applicable to British practice, but others are useful.

"**Architects' Detail Sheets.**" Edited by E. D. Mills. (London: Iliffe & Sons, Ltd. Price 30*s*.)

THIS is the fourth selection to be published in book form of detail sheets from "The Architect & Building News". It comprises 96 scale drawings of architectural details and more than a hundred photographs of work in Great Britain and other countries. The details relate to fireplaces, fittings, staircases, walls, windows, and other features. Nearly all of the designs are unusual (called here "contemporary"). In many cases it would have been courteous to give the names of the engineers concerned with the work.

"**German Standard Code of Practice DIN. 4224 : Design of Reinforced Concrete.**" (Berlin: Wilhelm Ernst & Sohn. Price 6 D.M.)

THE methods of design recommended in this new code are considered in "Der Stahlbetonbau," by Karl Kersten, which was reviewed in this journal for June 1958.

"**Vorgespannter Beton.**" By A. Mehmel. (Berlin: Springer-Verlag. Price 17.40 D.M.)

THIS work on prestressed concrete is based on the author's lectures at the Technische Hochschule, Darmstadt. Constructional details of the more important systems are discussed, and notes are given on methods of supporting and fixing the steel and ducts, the tensioning procedure, forms of anchorage, and the protection of the steel against corrosion.

There are fully-worked numerical examples for a continuous beam for a footbridge with post-tensioned steel and for a roof beam with pre-tensioned steel.

"**Rigid Frame Formulas.**" By A. Kleinlogel. (London: Crosby Lockwood & Son, Ltd. Price 90*s*.)

IN this translation of the new (twelfth) edition of the well-known German "Rahmenformeln" eleven frames have been replaced by eleven new frames, including symmetrical triangular frames with tie-rods and various conditions at the ends of the diagonals, symmetrical and unsymmetrical fixed rectangular frames with hinged joints, and frames with hinged or fixed bases and with or without ties at various heights. Small improvements have been made in the remainder of the book, but apart from the new frames the contents are substantially as before.

Books Received.

"Barragens Abóbada: Fundações, Projecto Sobre Modelos e Observações dos Protótipos" (Results of investigations of the foundations of arch dams). By Manuel Rocha and J. Laginha Serafim.

"A Utilização de Modelos no Dimensionamento das Estruturas; Aplicações Diversas" (A comparison of analytical and experimental methods of design). By Manuel Rocha and J. Ferry Borges.

"Solos das Estradas" (Portuguese report presented to the International Road Congress Istanbul, 1955).

[Lisbon: Laboratorio Nacional de Engenharia Civil. Printed in the Portuguese language, with brief summaries in English. No prices stated.]

"**Spannungen an Tunnelöffnungen mit rechteckigem Nutzquerschnitt und kreisbogenförmiger Überwölbung**" (Stress Distribution Around Tunnel Openings). By Rudolf Hiltscher.

"**Inverkan av deformationer i väggar på bjälklagslaster och bjälklagsdeformationer vid flervåningshus**" (Effects of Deformations of Walls in Multiple-Story Houses). By Av Henrik Nylander and Einar Eriksson.

"**The Design of Reinforced Concrete.**" By Carl Forssell. [Stockholm: Royal Institute of Technology. No price stated.]

"**Investigations of Bond between Reinforcement and Concrete.**" By P. O. Jonsson. [Stockholm: Swedish Cement and Concrete Research Institute. Price 8 kroner. Printed in English.]

"**Types of Road Surfacing and Maintenance using Tar or Asphaltic Bitumen.**" Road Note No. 5. (London: H.M.S.O. Price 3*s*.)

"**Field Studies of the Movement of Soil Moisture.**" Technical Paper No. 41. (London: Department of Scientific and Industrial Research. Price 6*s*.)

Hangar with Prestressed Roof at Abingdon.

ROOF CAST ON THE FLOOR.

A METHOD not hitherto used in Great Britain is being used for the erection of the shell roofs and supporting columns for a hangar at Abingdon, Berks, for Transport Command of the Royal Air Force.

The requirements were set out by the Air Ministry, and selected contractors were invited to submit designs and tenders for construction in steel, aluminium alloy, or any type of concrete construction. Of six tenders submitted, one was for aluminium construction, one for structural steel with an aluminium roof, one for prestressed precast concrete, and the others for prestressed concrete cast in place. The design described was selected because it was considered to satisfy the requirements of stability, appearance, time of erection, durability, and low maintenance costs, and was also 10 per cent. cheaper than the next lowest tender and 28 per cent. cheaper than the highest tender.

The hangar is divided into three bays, each of which has an area of 186 ft. by 110 ft., and is roofed by three barrel vaults supported by a column at each corner; the bays are connected by two link-bays each 36 ft. wide. The total uninterrupted floor area is 613 ft. by 103 ft.

The hollow edge-beams are 6 ft. 5 in. wide, 5 ft. 1 in. deep at the outer face and 10 ft. deep at the inner face. The bottom is 6 in. thick and is prestressed with six cables; the outer wall is 5 in. thick and prestressed with eight cables; the inner wall is 5 in. thick and prestressed with nine cables; each cable comprises twelve wires of 0·276 in. diameter. The solid valley beams are 9 in. wide by 10 ft. deep, and are prestressed with fourteen cables each containing twelve wires of 0·276 in. diameter. The gable beams are prestressed with sixteen cables each containing twelve 0·276-in. wires; they are of inverted tee-shape 18 ft. deep, 9 in. wide at the stems, and 2 ft. 6 in. wide at the bottom for a depth of 1 ft. 6 in.

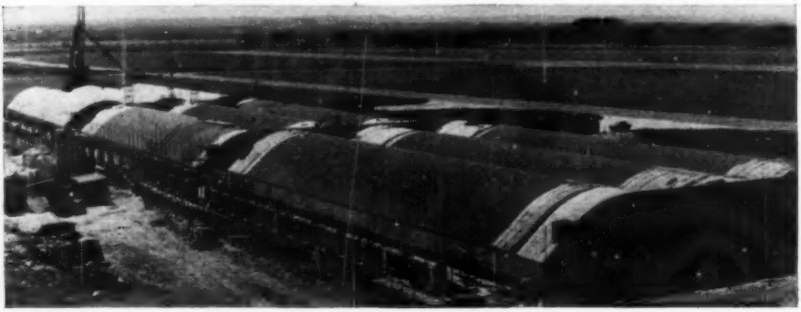


Fig. 1.—The Roof During Casting.

The beams were prestressed from both ends, and a "walkie-talkie" radio was used to co-ordinate the work.

In each "shell" there are 106 Freyssinet cables, each comprising twelve 0·276-in. wires. The roofs are $3\frac{1}{2}$ in. thick at the crown and 5 in. thick at the valleys.

The roofs were cast on the floor of the hangar, on which building paper was laid to prevent adhesion of the beams which were cast directly on the floor. Formwork travelling on bogies was used for casting each vault in several sections; as each section was completed the formwork was moved on to the next section, the completed sections resting on the outer beams and the valley beams. *Fig.* 1 shows two of the bays completed and the third being cast. A section through the roof is shown in *Fig.* 2.

Each roof weighs about 1400 tons, and

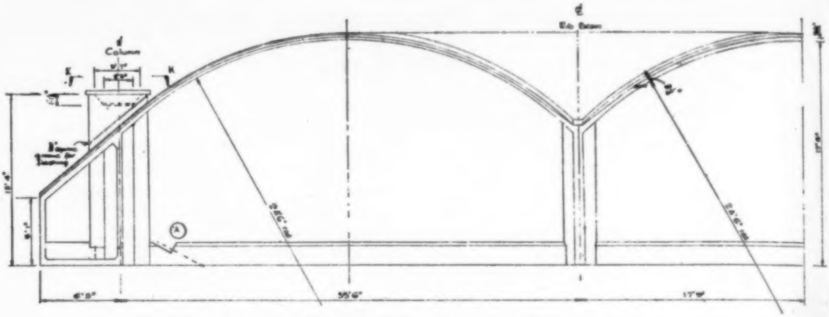


Fig. 2.—Part Cross Section Through Roof.

the method of lifting a bay and forming the columns is shown in *Figs.* 3 to 6. The columns are formed of precast T-shape blocks of reinforced concrete to form a hollow column 4 ft. 6 in. by 3 ft. 6 in. in cross section; 1000 of these blocks, each weighing half a ton, were precast on the site for the twelve columns of the hangar.

The roofs were first lifted to a height of 2 ft. 8 in. above ground level by means of hydraulic jacks at each side, care being taken to ensure that the roof did not deviate more than 1 in. from the level position ; this operation took six hours, after which the blocks were inserted to form the columns as the roof was raised to its final height of 46 ft. The hydraulic jacks each had a capacity of 200 tons.

When the roof had been raised high enough to accommodate another course of blocks, two of the jacks were locked and the other two retracted while two blocks were placed in position on opposite sides of the column. Before the blocks were inserted a layer of stiff 1 : 3 cement mortar was placed on top, the depth of $\frac{5}{8}$ in. being ensured by the use of a steel tray fixed to the top of the block. The jacks that had been retracted were then set under the new blocks and the lifting continued until the roof was high enough for the insertion of blocks at the other

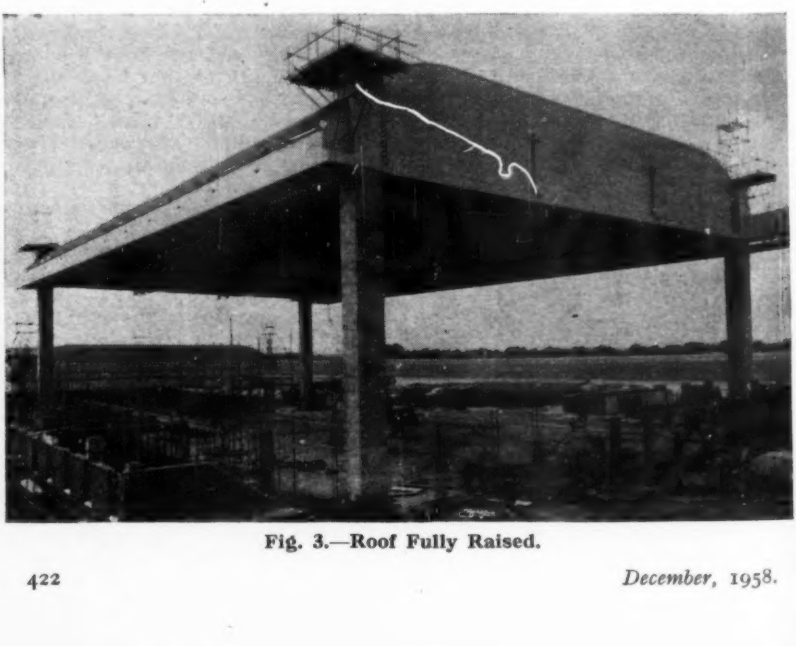


Fig. 3.—Roof Fully Raised.

two sides. When the load was applied the mortar joints were compressed to a thickness of ½ in., and none of the mortar was squeezed out.

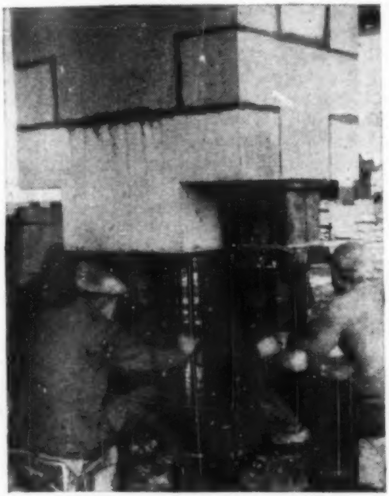


Fig. 4.—Raising a Column.

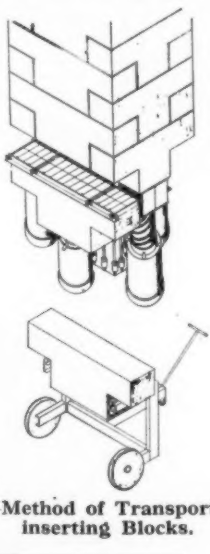


Fig. 5.—Method of Transporting and inserting Blocks.

The columns were prestressed by fourteen Macalloy bars of 1⅛ in. diameter, anchored to a foundation block below ground level and tensioned from the top after the space between the blocks had been filled with concrete. In *Fig.* 6 is shown the steel bracing used to stabilise the structure before tensioning the bars.

In the design a maximum wind velocity of 85 miles per hour was allowed for, and a maximum variation in temperature of 45 deg. Fahr. The compressive strengths of the concrete were specified to be 4000 lb. per square inch at 28 days in the shells and 6000 lb. per square inch in the beams.

The front of the hangar will have sliding folding doors operating in three pairs. The roof is insulated with boarding and roofing felt to retain the heat provided by radiant heating panels suspended from the roof and walls.

The scheme was evolved by the contractors, Messrs. John Laing & Son, Ltd., in collaboration with Messrs. Ove Arup & Partners, the consulting engineers. It is estimated that this method of erecting the roof was cheaper due to the saving of scaffolding and plant that results from casting the roof and installing the services and finishes at ground level. Each bay was raised in one week by thirty men.

Civil Engineering Scholarships.

The first awards have been made by the Civil Engineering Scholarship Trust to eleven candidates. The value of the scholarships is an average of nearly £400 a year, and will enable the boys to commence a three-years' or four-years' course of civil engineering at a university. The Trust received 117 applications for the scholarships, and expects to increase the number available next year. The Trust is sponsored by the Institution of Civil Engineers, the Association of Consulting Engineers, and the Federation of Civil Engineering Contractors.

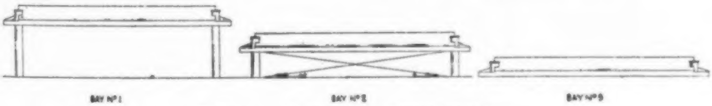


Fig. 6.—Stages of Construction.

A Concrete and Glass Canopy.

A CONCRETE and glass canopy has been constructed at the Savoy Hotel, London, as part of a porte-cochére. It has an area of 2500 sq. ft. and comprises three arched bays; the spans of the side arches are 15 ft. 6 in. with a rise of 1 ft. 9 in. and the span of the central arch is 20 ft. with a rise of 2 ft. 3 in. (*Fig.* 2). At each end of the canopy a curved reinforced concrete slab, 4½ in. thick, cantilevers upwards.

The arches are supported on inverted tee-beams the bottom flanges of which are curved to accord with the concrete and glass panels. The span of the beams is 32 ft.; they are supported at one end on bearings cut into the building. The outer supports consist of four circular columns which taper from 15 in. at the top to 12 in. at ground level. The height from the ground to the soffit of the beams is 16 ft. 9 in. The bases of the columns are of reinforced concrete, and are connected by a beam 15 in. square about 6 in. below the ground. All the concrete for the foundations and superstructure was cast in place.

The architects were Messrs. Thurlow, Lucas & Janes, the contractors were Lenscrete, Ltd., and the structure was designed by the British Reinforced Concrete Engineering Co., Ltd.

Fig. 1.

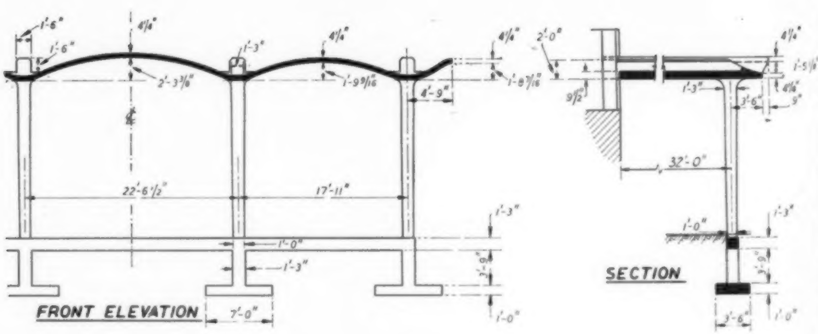


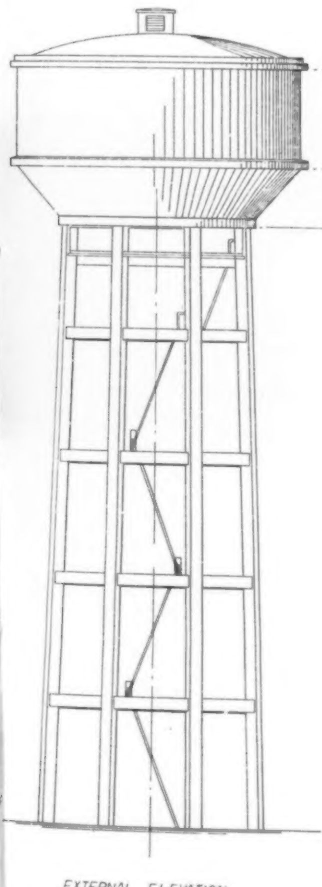
Fig. 2.

Large Water Tower at Dunstable.

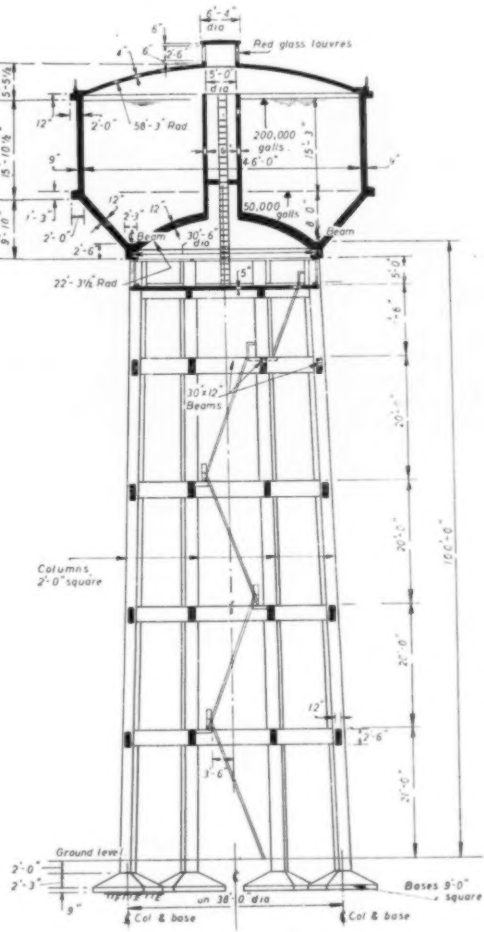
A WATER tower with a capacity of 200,000 galls. and a height of more than 120 ft. is being built at Dunstable for General Motors, Ltd. It will be one of the largest in the country.

The tank is of the Intze type, supported on eight reinforced concrete columns equally spaced on a pitch circle 38 ft. in diameter. The foundations are independent bases resting on chalk, which occurs 2 ft. below ground level. The underside of the tank will be 100 ft. above the ground and the columns will be braced at intervals of 20 ft. A valve platform consisting of a floor-slab 5 in. thick supported on beams will be provided 7 ft. 6 in. below the underside of the bottom of the tank.

The roof will be a dome with a thickness of 4 in. and a radius of 58 ft., and at



EXTERNAL ELEVATION



SECTIONAL ELEVATION.

the crown there will be a cupola with a diameter of 5 ft. 8 in. and a height of 3 ft. The roof will be connected by a ring-beam to the cylindrical part of the tank, which will have an overall diameter of 47 ft. 6 in., the wall being 16 ft. high and 9 in. thick. A lower ring-beam will connect this wall to the conical part of the bottom of the tank, which will be 12 in. thick and 10 ft. high, and will balance the domed floor. The domed bottom will have a thickness of 12 in. and a radius of 22 ft. 3 in. The main ring-beam will be at the line of intersection of the conical and domed tank-bottom and will transmit the load to the supporting columns.

A valve-pit below ground between the column bases will contain the valve controls and pipes for the tower. It is 14 ft. 6 in. long, 14 ft. wide, and 8 ft. 6 in. high, and is of reinforced concrete; the walls and roof are 9 in. thick. The floor is 12 in. thick and projects 12 in.

The tower was designed by the British Reinforced Concrete Engineering Co., Ltd., who also supplied the reinforcement, and the contractors are Messrs. Fletcher & Co., Ltd.

The Construction of Roads on Peat.

Technical Paper No. 40 of the Department of Scientific and Industrial Research is entitled "A Review of Existing Methods of Road Construction over Peat," and is available from H.M. Stationery Office at 3*s*. 6*d*. It has been prepared at the Road Research Laboratory, and gives brief descriptions of methods that have been used in the United Kingdom and elsewhere for the construction of roads over peat. The following notes are given on some concrete roads laid on peat.

Since concrete roads are usually about half the thickness of flexible roads their smaller weight should normally result in less settlement. In addition, concrete roads carrying medium traffic over peat tend to develop less abrupt longitudinal undulations than do flexible pavements. Cracking and severe loss of shape may eventually occur, however, especially if traffic increases.

In Northern Ireland a common practice is to lay doubly-reinforced slabs 8 in. thick directly on the peat or on a base of material taken from old waterbound macadam roads. The slabs are usually about 50 ft. long, 10 ft. or 20 ft. wide, and dowelled at the joints. These roads usually behave reasonably satisfactorily under moderately light traffic. Longitudinal waviness is common, but relatively little cracking occurs. The importance of dowelled joints or of coastructing slabs to full width of the road is shown by the fact that differential settlement of adjacent slabs usually occurs when dowel bars are omitted. The deformation of some of these roads has, however, become sufficient to warrant total reconstruction, although many still continue to give satisfactory service.

In East Anglia reinforced and unreinforced concrete have been used for minor roads over peat. In general, the performance of these roads has been good. Roads in use up to thirteen years showed that unreinforced slabs 8 in. thick were particularly free from trouble if their length was limited to 12 ft.

In Scotland concrete on main roads with medium traffic over peat has given reasonably satisfactory results. On the Perth–Inverness road an 8-in. doubly-reinforced surface was laid, with reinforced beams 18 in. deep under the edges and joints. No noticeable settlement of this road has occurred after 26 years. The concrete road at Spean bridge on the Fort William–Inverness road is doubly reinforced and 8 in. thick, thickened to $15\frac{1}{2}$ in. at the outside edges; the road has settled considerably, although its riding qualities are reasonably good. A concrete road between Newtonmore and Laggan Bridge is doubly reinforced and 8 in. thick; it settled so unevenly that it was covered with a bituminous surfacing. In the only known case in which concrete was used over peat carrying frequent and heavy traffic the road was in frequent need of repair.

Petrol Storage Tanks in London.

FOUR light gasworks feedstock tanks, for the storage of petrol, are now being built for the South Eastern Gas Board at Greenwich, Wandsworth, Croydon, and Sydenham. The capacities of the tanks at Greenwich and Croydon will be 150,000 galls., at Sydenham 100,000 galls., and at Wandsworth 240,000 galls. in two compartments.

In order to reduce the risk of fire, the tanks are below ground and are designed to conform to the requirements of the London County Council; these require the tanks to be of steel surrounded by high-quality concrete, and to be tested with a head of water 1 ft. greater than the depth of the tank. It was also considered desirable that the bottom of the excavation should not be below the level of the ground-water, and the restricted sites available necessitated the use of rectangular tanks. The time of construction was required to be as short as possible.

In the interest of economy all the tanks are of similar construction, the only differences being those due to the capacities required and the configuration of the sites. The following description relates particularly to the tank at Greenwich (*Figs.* 2, 3, 5 and 6); that at Wandsworth is shown in *Figs.* 1 and 4.

The general arrangement is shown in *Fig.* 2. In order to allow the steel lining to be tested for leaks, the bottom plates were placed on a steel grillage which rested on a prestressed concrete slab at the bottom of the excavation. A layer of tarred sand, in which drainage channels were formed (*Fig.* 1), was placed on top of the concrete base. The steel side-plates were subjected to the full test load before the surrounding concrete was placed, and were therefore arranged as shown in *Fig.* 3 to provide the necessary resistance to bending. The steel plates forming the roof are attached to the underside of prestressed beams, by means of stiffeners cast in place, on an assembly bay adjacent to the tank (*Figs.* 4 and 5), and were then moved into position on rollers. The welding was completed, the floor of the tank tested, jacks inserted under each pair of roof beams, and the tank lifted in order to remove the grillage; the structure was then lowered so that the roof beams were supported on previously-prepared side beams and the floor was supported on the prestressed base. The full test load was applied, and the walls concreted.

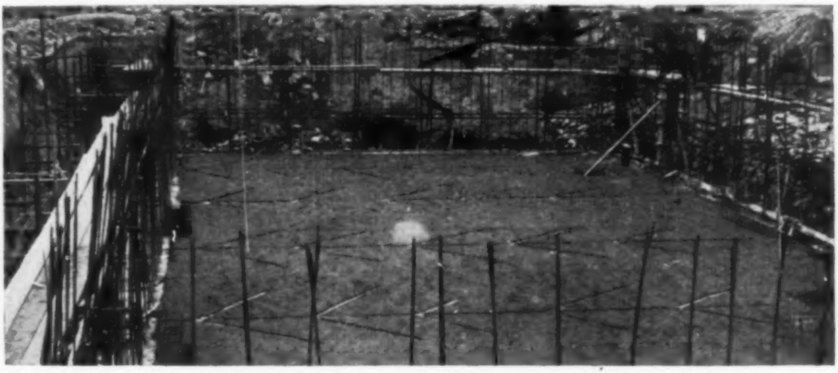
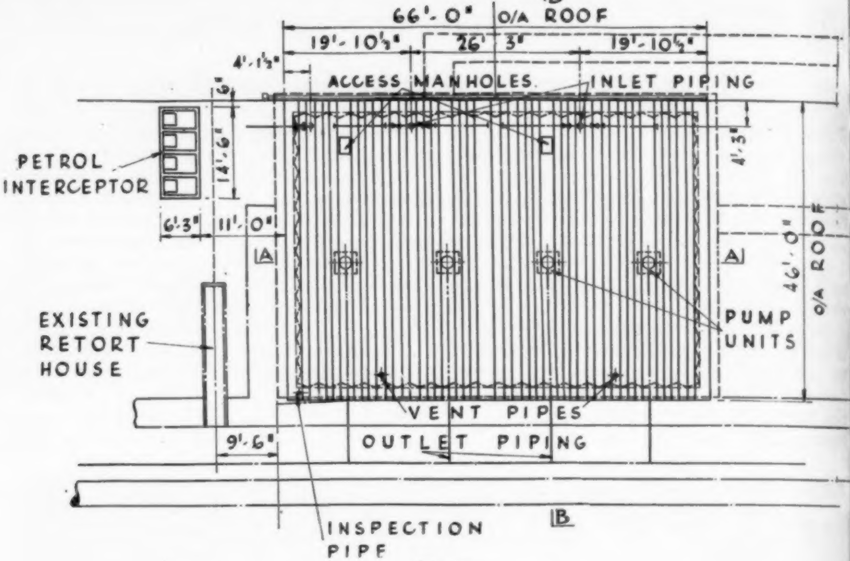


Fig. 1.—Draining Channels in Base.

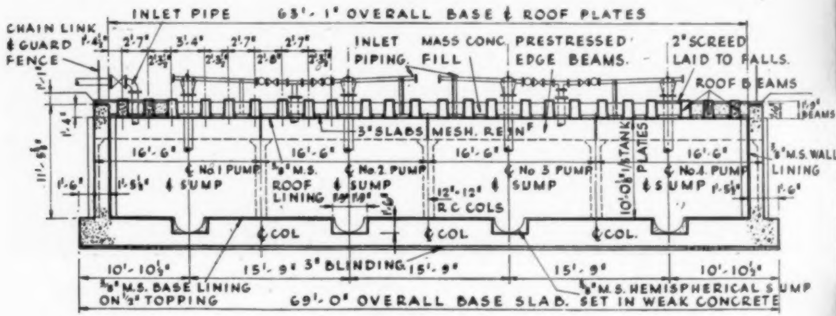
The sequence of construction is given in the following.

(1) The pit was excavated and the prestressed slab laid at the bottom. Concrete columns supporting the prestressed side beams were then cast along two sides of the pit.

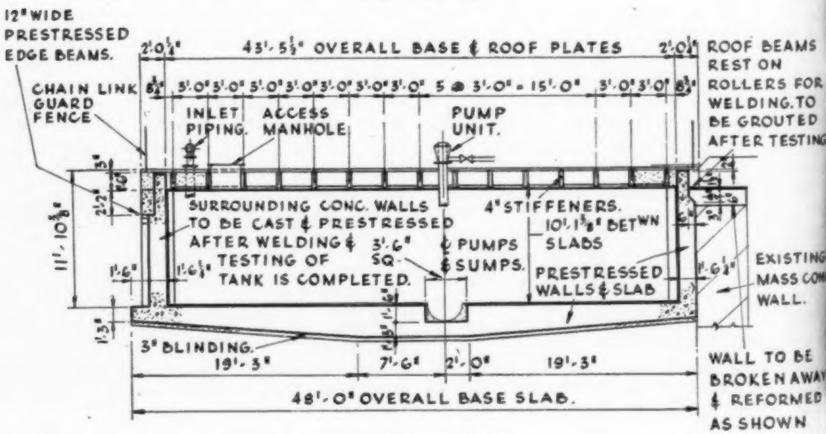
(2) The steel grillage was placed on the base slab, and the floor and walls of the steel tank erected.



PLAN.



SECTION A–A.



SECTION B–B.

Fig. 1.

Fig. 3.—Erecting the Steel Tank.

Fig. 4.—Construction of Roof.

Fig. 5.—Moving Roof Units into Position.

(3) At the same time the precast prestressed roof beams were placed in pairs on the roof plates. Stiffeners were placed between each pair of beams; the reinforcement of the stiffeners passes through holes formed in cleats attached to the plates to ensure that these are firmly connected to the beams.

(4) The roof units were moved along runways consisting of steel sheet piles to the beams alongside the tanks, and then along these beams into position. A special type of roller known as a "mover skate" was used. The longitudinal and peripheral welds in the roof were made, and the steel tank thereby completed.

(5) Jacks were placed under the ends of the roof units, and the tank lifted until the grillage could be removed. During this operation, temporary suspension rods supported the steel bottom of the tank. The tank was then lowered until the floor rested on the base slab, and the roof beams were supported on the concrete beams at the side (*Fig.* 6).

(6) The tank was tested, and the surrounding concrete placed and prestressed vertically. Finally the roof was covered with concrete and the ground was backfilled.

The work was designed by the Central Construction Department of the South Eastern Gas Board, the general contractors are the Demolition and Construction Co. Ltd., and the steel tank was fabricated by the Redheugh Iron & Steel Co., Ltd.

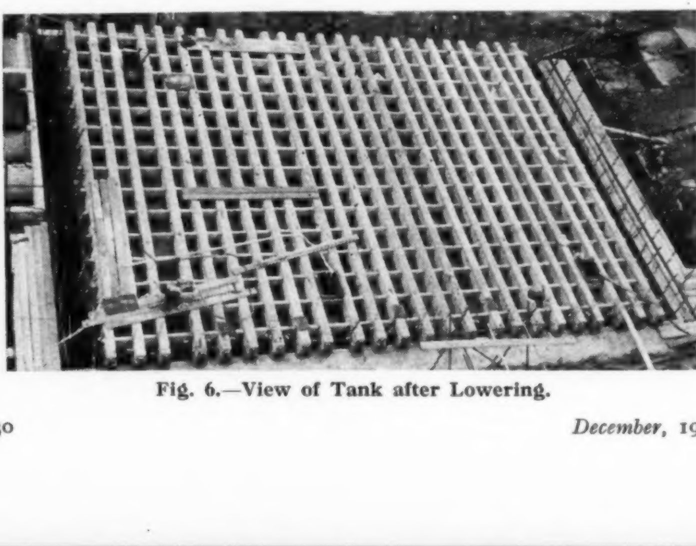


Fig. 6.—View of Tank after Lowering.

A Swiss System of Prestressing.

APPLICATION TO A BRIDGE IN KENT.

THE bridge described in this article is one of the first structures in Great Britain to be prestressed by the BBRV system of post-tensioning, which was developed some years ago in Switzerland and has since been used elsewhere (the initials BBRV are those of Messrs. Birkenmaier, Brandestini, Rôs, and Vogt, the inventors of the system). It differs from other systems in that multiple-wire cables capable of sustaining working loads exceeding 100 tons are used, and each wire is anchored at each end by means of enlarged heads formed on the wire. Details of a typical cable to provide a force of 90 tons are given in *Fig.* 1.

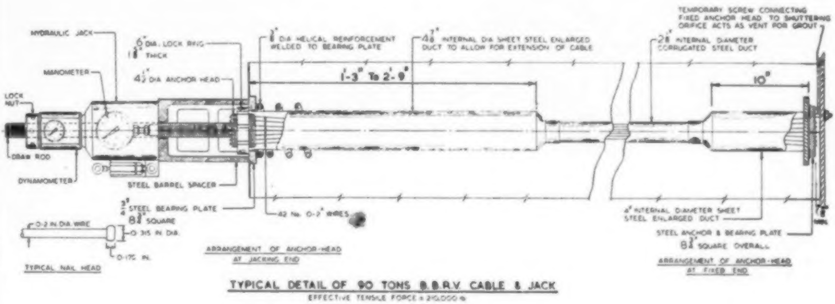


Fig. 1.

The Cables.

A cable may comprise up to 44 wires the number depending on the working load, which for standard cables may be 28, 56, 90, or 125 tons ; a cable for a working load of 250 tons is being developed. The space occupied is small in relation to the force in a cable. All wires are cut to the same predetermined length, two steel anchor-heads (each of which is perforated to accommodate all the wires) are threaded over the ends of the wires, enlarged heads are formed at both ends of each wire, and the cable is then ready to be placed in position.

The wire used in this country is generally hard-drawn, with a tensile strength from 100 tons to 110 tons per square inch in accordance with B.S. No. 2691 and with a diameter of 0·20 in. or 0·276 in. In order that the enlarged head, called a "button-head" or "nail-head", may be formed properly, the carbon content of the steel should be about 0·7 per cent. The shape and proportions of the head are also important (*Fig.* 1) ; tests show that a diameter of about 1½ times the diameter of the wire and a thickness of about seven-eighths of the diameter are most satisfactory because the strength of the head so formed exceeds the tensile strength of the wire. The head is formed by cold-upsetting of the end of the wire in a machine (*Fig.* 2) operated by electrical or hydraulic means. The wire is placed in a slot in the machine, the slot is closed to grip the wire, and a crank-operated ram pressing on the end forms the required shape. The process, which occupies only a second or so, is generally carried out near the bench on which the cables are formed ; alternatively the heads may be formed on one end of each wire and a portable machine used to form those on the other ends after the cable is placed in position.

It is usual for the cable to be made up completely with a jacking anchor-head at one end and a fixed anchor-head at the other end, the wires being encased in a sheath with enlarged ends as shown in *Fig.* 1. The complete cable is fixed in the shuttering before concreting. When the concrete has hardened the cables are tensioned and grout is injected in the sheaths.

The anchor-head at the jacking end of the cable is a steel annulus (*Fig.* 3) perforated with two concentric rings of holes to receive the wires ; a larger central hole is tapped to receive a threaded draw-rod. The outer rim of the annulus is threaded to receive a locking ring which transmits the prestressing force to a steel plate bearing against or embedded in the concrete. If the cable is completely made up before insertion in the shuttering, the fixed anchor at the end remote from the jack comprises a square steel block and bearing plate (*Fig.* 1) without a central hole but perforated to allow the wires to pass through and be retained by the heads.

The sheaths are up to 2⅛ in. diameter. The enlarged ends are up to 4 in. diameter at the fixed end and 4⅞ in. diameter at the jacking end. The length of the enlargement at the jacking end is sufficient to allow the anchor-head to lie inside the sheath before tensioning the cable.

Long cables are formed by coupling together two or more cables, on the meeting ends of which jacking anchor-heads are provided, a short threaded rod being screwed into both anchor-heads. It is desirable to tension curved cables and long cables from both ends, and a jacking anchor-head is provided at each end.

An alternative method is to insert the cables in ducts formed in the concrete. The anchor at the fixed end comprises a steel annulus similar to the anchor-head at the jacking end except that it is smaller in diameter because of the absence of the central hole for the draw-rod. A modification of this method is to form the nail-heads on one end of the wires, insert the wires in an anchor-head of the smaller type described, insert the wires and anchor-head into the duct from the fixed-end, thread a jacking anchor-head over the free ends of the wires, and finally form the nail-heads on these ends.

Fig. 2.—Machine for Forming Heads on Wires.

The ducts in which the cables are inserted are formed by one of the usual methods. The diameter of the duct is between 1⅜ in. and 2⅜ in., depending on the number of wires in the cable. The duct is enlarged at the fixed end to between 2¼ in. and 4⅞ in., to allow for the spreading of the wires as they enter the anchor-head, and to between 2⅞ in. and 5¾ in. at the jacking end to allow the anchor-head to be a short distance inside the duct before the cable is tensioned. If completed cables are inserted the ducts must be large enough throughout to allow for the passage of the annular fixed-end anchor-heads. If the nail-heads are formed on one end of the wire after it is inserted in ducts, the length of the enlarged duct at the fixed end is increased so that the fixed-end anchor can enter the duct sufficiently far to enable the nail-heads to be formed at the other end.

Fig. 3.—Anchor-head at Jacking End.

Tensioning the Cables.

The cables are tensioned by an hydraulic jack with a capacity of 100 or 150 tons. The jack is operated manually by a lever on the body (*Fig.* 4), and can also be adapted for operation from a distance by a high-pressure pump operated manually or electrically. A threaded draw-rod is screwed into the central hole in the anchor-head. The jack has a hole centrally through the body so that it can be slipped over the draw-rod in which position it is retained by a lock-nut on the protruding

end of the draw-rod. A steel barrel acts as a spacer between the jack and the bearing-plate.

The jack pulls the draw-rod, bringing the anchor-head with it and thereby tensioning the wires. When the required force or extension is obtained, the locking-ring is screwed tightly against the bearing-plate; the final tightening is by means of a hammer and a bar, the end of the bar engaging in slots in the periphery of the locking-ring. The anchorage of the wires can be inspected at any time during the operation and a broken wire can be detected before grouting.

The extension is indicated by graduations on the ram of the jack. The force is indicated by the manometer attached to the jack or more accurately (within ± 1 per cent.) by means of a dynamometer (*Fig.* 5). The losses due to friction, shrinkage, and creep can be allowed for by attaching the dynamometer and operating the jack at any time after the initial tensioning so long as the ducts have not been grouted. It is usual for these losses to be made good immediately before grouting, by re-tensioning the cable to the initial force assumed in the design. There is no loss due to slipping of the wires at the anchorage.

Application to a Service Bridge.

The bridge shown in *Fig.* 6 and illustrated in course of construction in *Fig.* 7

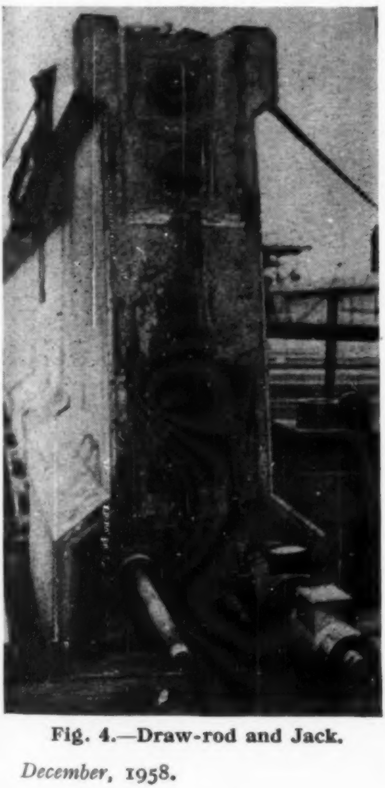


Fig. 4.—Draw-rod and Jack.

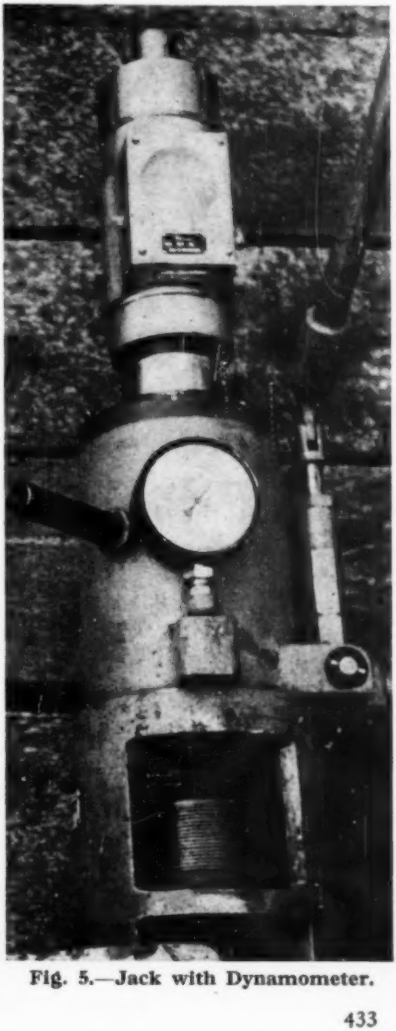


Fig. 5.—Jack with Dynamometer.

supports services at the works of the South Eastern Gas Board at Isle of Grain, Kent, and comprises two spans of about 95 ft. and one of about 34 ft. Each span comprises two prestressed longitudinal main girders of I-section supporting precast transverse beams carrying precast deck slabs. The ends of the transverse beams rest on the bottom flanges of the main girders and are secured by two tensioned 0·276-in. wires anchored in the girders. The girders are supported on prestressed double-cantilever cross-heads carried on reinforced concrete frames as shown in *Figs*. 6 and 7. One end of each main girder is fixed and the other carried on a roller bearing in the longer spans, or a sliding bearing in the shorter span. The main girders are made with precast blocks (*Fig*. 8) which were assembled in position on the temporary steel Bailey bridge shown in *Fig*. 7. The meeting faces of the blocks were bush-hammered and after erection and alignment the joints were filled with mortar comprising 1 part (by volume) of cement to 2 parts of sand ; the water content was about 6 gall. per 112 lb. of cement. The cables were not tensioned until the grout attained a compressive strength of more than 2000 lb. per square inch.

Each of the longer girders has five cables, two of which have a nominal capacity of 50 tons and three of 100 tons. The nail-heading at the jacking ends was done after the cables had been inserted in the ducts ; the enlarged part of the duct at the fixed end was therefore longer than usual, so that the anchor-head could pass inside the duct to enable the wires at the jacking end to be pulled sufficiently far out of the duct to enable the heads to be formed on the ends. The three cables in the bottom flange are contained in an elliptical duct, thereby avoiding the reduction of the area of concrete which would occur if three large circular ducts were provided. The profiles of the cables are shown in *Fig*. 9 and the positions at the end of the girder in *Fig*. 4.

The cables were tensioned with the jack operating at the central support and in the following sequence : No. 1 (lower curved 50-tons cable) ; No. 2 (upper curved 50-tons cable) ; No. 3 (outer straight 100-tons cable) ; No. 4 (inner straight 100-tons cable) ; and No. 5 (central 100-tons cable). At the stage shown in *Fig*. 4, cables Nos. 1 and 2 have been tensioned, locked, and grouted ; cable No. 3 has been tensioned and locked, but the draw-rod is still in position ; cable No. 4 is being tensioned, the jack being in position ; cable No. 5 has not

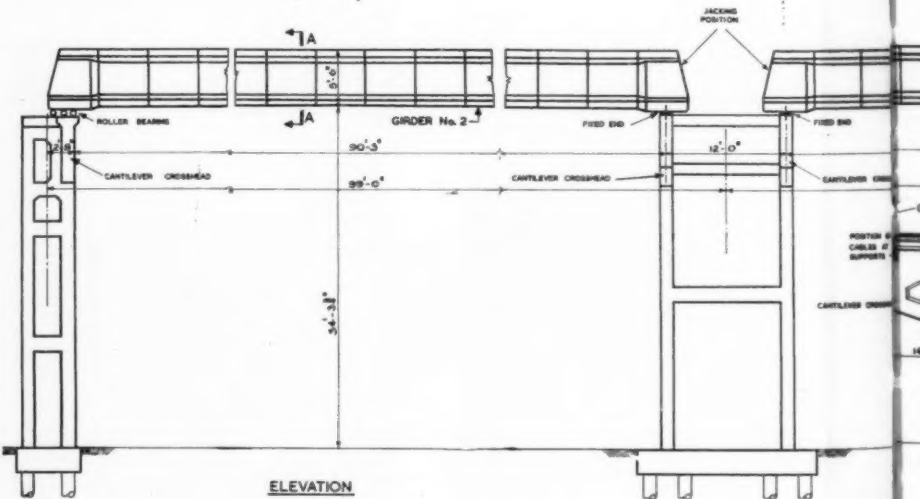


Fig. 6.—Prestressed

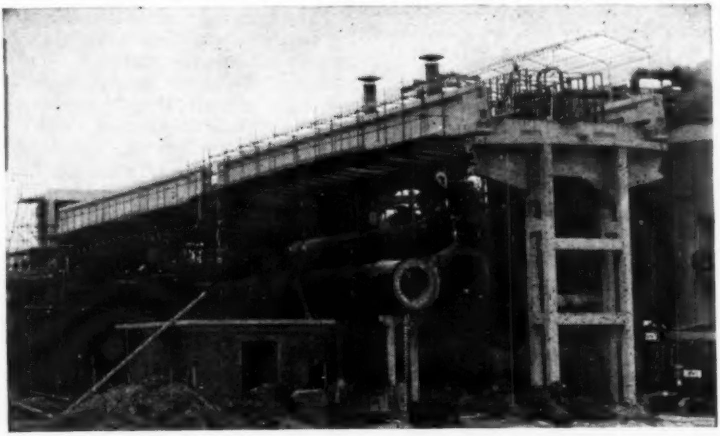
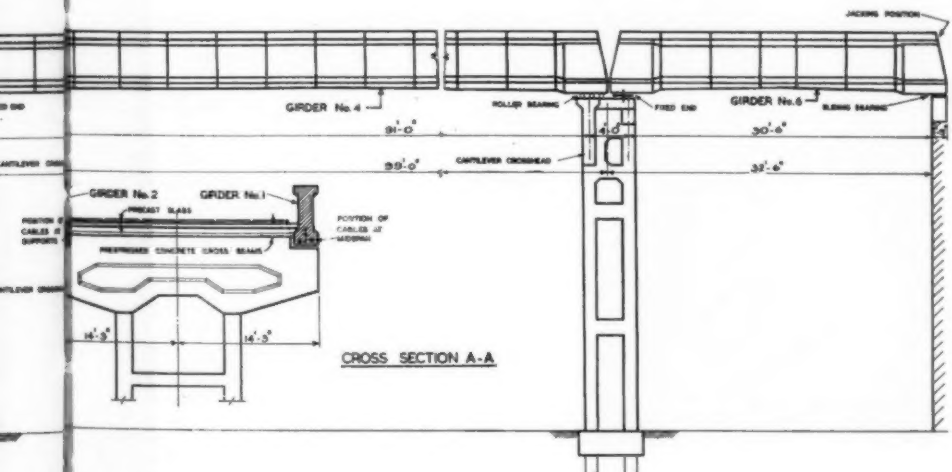


Fig. 7.—Prestressed Concrete Service Bridge: two Spans during Construction.

yet been tensioned, and the anchor-head is visible just inside the enlarged part of the duct.

Each 50-tons cable contains twenty-six wires, and each 100-tons cable forty-two wires, all of 0·2 in. diameter. The average tensile strength of the wires is 112 tons per square inch and the 1 per cent. proof stress not less than 98·8 tons per square inch. The initial tensile stress in the wires was about 170,000 lb. per square inch (about 5300 lb. per wire) in each straight cable and an effective force of about 5000 lb. per wire in the curved



...crete Girder Bridge.

cables. Before grouting, the cables were re-tensioned to about 5000 lb. per wire to make good the losses; the grout was similar to the liquid mortar between the blocks, but it also contained a chemical to retard setting and a non-shrinking agent. The South Eastern Gas Board is contemplating the use of a bituminous liquid such as gas-works tar, instead of a cementitious grout, for injection into ducts to ensure greater protection of the wires.

After the tensioning and grouting operations were completed, the anchor-heads were embedded in concrete in the recesses at the ends of the girders and held in place by small grids of mild steel bars (*Fig.* 10) welded to the bearing plates.

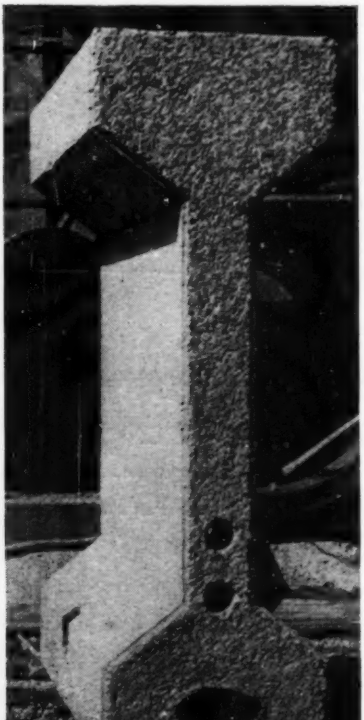


Fig. 8.—Precast Block with Bush-Hammered Face, showing Cable Ducts.

Stresses during Prestressing.

The sequence of tensioning the cables is important to ensure that no significant tensile stresses are produced. The stresses are not easy to determine because of the variable contribution of the dead load; the weight of the precast blocks forming the girder caused the temporary steel bridge to deflect, and when the first cables were tensioned the weight on the middle of the steel bridge was reduced. Consequently the elastic deflection of the steel bridge was reduced so that it still supported some of the weight of the blocks, even at the middle of the span. All the weight was transferred entirely to the end supports of the girder only when the last cable was tensioned.

As tensioning of the cables proceeded observations were made of the degree of freedom of the blocks forming girder No. 2. It was found (*Fig.* 10) that all the blocks were supported on the steel bridge when cables Nos. 1 and 2 were tensioned; a few central blocks were free when cable No. 3 was tensioned; and all but a few blocks at each end were free when cable No. 4 was tensioned. The diagrams on *Fig.* 11 show the probable stresses at an end joint and at a joint near the middle of the span during the tensioning operations. Comparison with the corresponding diagrams, assuming all blocks to be free of the steel bridge, show that in some cases the stresses are likely to be the reverse of what might be expected.

Reduction of Prestress due to Friction.

Some observations were made to determine the loss due to friction between the curved cables and the sides of the ducts. In particular the loss in the upper cables in three of the double-cantilevered crossheads at the supports (*Fig.* 9) was investigated. The radius of these cables is about 26 ft. and the length about 30 ft.; the ducts comprise corrugated steel tubes of 1⅝ in. internal diameter. The two lower cables (Nos. 2 and 3) turn through an angle of about 55 deg. and the upper cable (No. 1) through about 47 deg. Each cable was tensioned with a force P_0 of about 58 tons with the jack and dynamometer attached at one end. The locking-ring at this end was then tightened, and the jack released and transferred with the dynamometer to the other

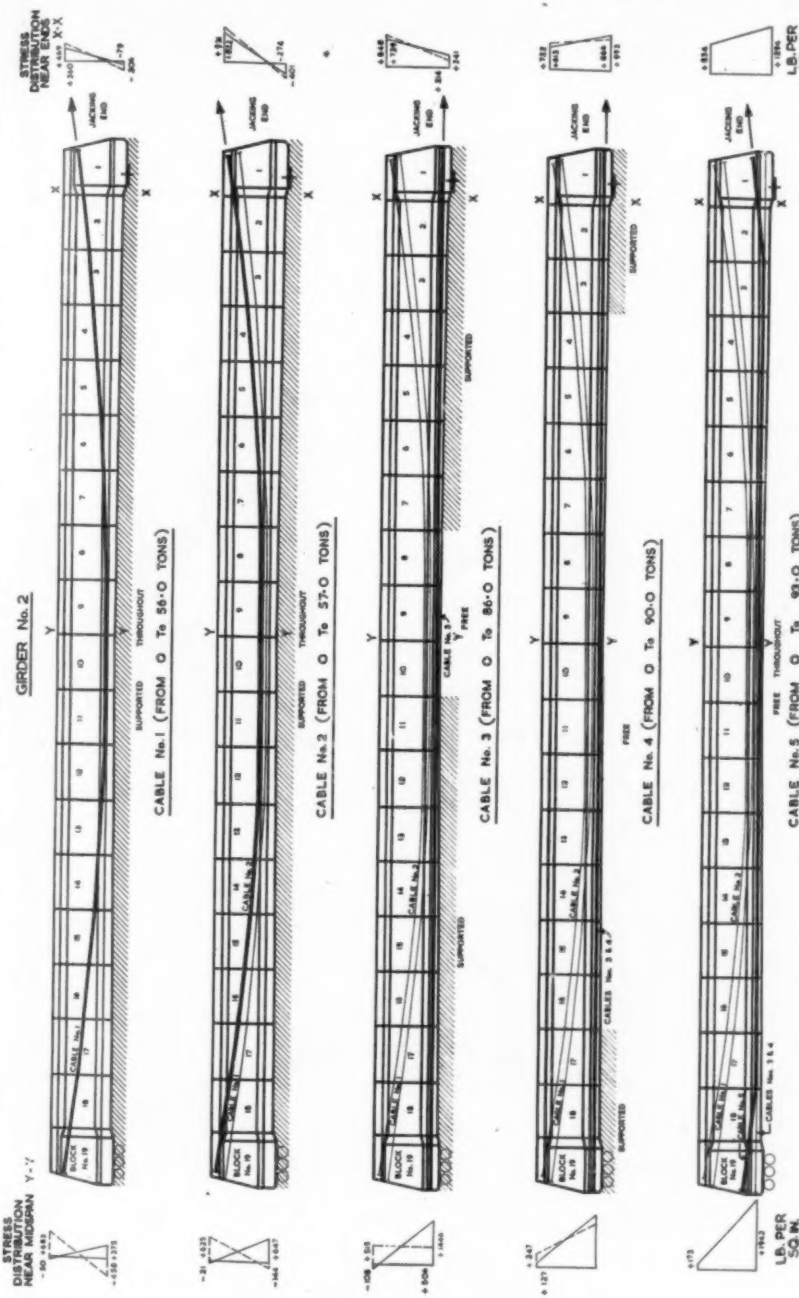


Fig. 9.—Profiles of Prestressing Cables.

TABLE I.

Cross head	Radius (ft.)	Angle α deg.	Angle α radians	Forces: Jacking force (tons) P_0	Forces: Dynamometer reading (tons) P_D	Reduction due to friction, Measured: (tons) $P_0 - P_D$	Measured: (per cent.) $\frac{P_0 - P_D}{P_0} \times 100$	Calculated: (tons) $P_0 - P_D$	Calculated: (per cent.) $\frac{P_0 - P_D}{P_0} \times 100$
A	25·85	47	0·820	58	45·8	12·2	21·0	12·7	21·8
B	25·85	47	0·820	58	46·6	11·4	19·5	12·7	21·8
C	25·85	47	0·820	57·7	44·9	12·8	22·2	12·7	21·8

end. A pull was applied to the anchor-head at this end, and at the instant when the locking-ring at this end became free the force P_D at this end was recorded. The difference between P_0 and P_D represents the reduction due to friction of the cable in the duct and the friction of the jack. Before removing the jack, this reduction was made good by tensioning the cable from this end to the full design load.

The observations are given in *Table* I, which also shows the theoretical reduction calculated in accordance with the exponential formula given in the British draft code for Prestressed Concrete and the draft Code for Liquid-retaining Structures. The formula can be expressed in a modified form, namely, that the reduction in tons due to friction at a distance of x ft. from the tangent point nearer the jacking end is $\left[1 - e^{\frac{-\mu x}{R}}\right]P_0$ tons, in

Fig. 10.—The Anchors.

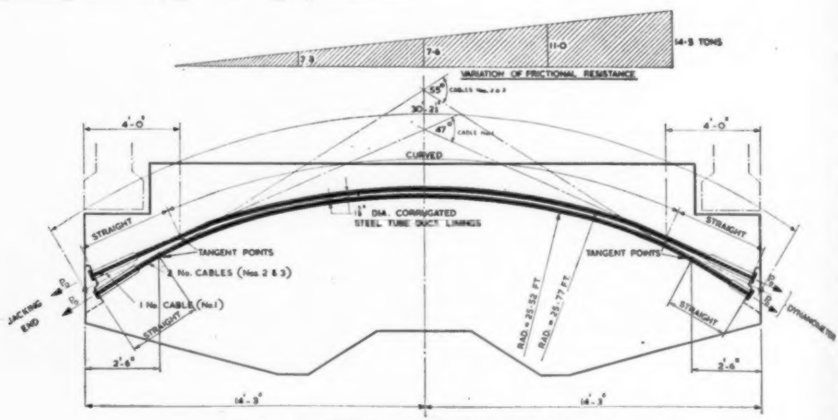


Fig. 11.—Effect of Tensioning Cables.

which P_0 is the jacking force in tons, R the radius of the cable (or inner edge of the duct) in feet, and μ the coefficient of friction which in this case is assumed to be 0·30 for steel wires in a corrugated-steel duct. For the total reduction $P_0 - P_D$ in the entire length of cable between the tangent points, x is the length of the curved part of the cable and therefore the term $\frac{x}{R}$ is equal to α radians, in which α is the angle through which the cable is deflected in the length x ft. Therefore

$$P_0 - P_D = \left[1 - \frac{1}{e^{\mu x}}\right] P_0 \text{ tons.}$$

Table I shows that the measured total reduction was between 20 per cent. and 22 per cent., whereas the reduction calculated at the time of the design of the structure was between 21 per cent. and 22 per cent. excluding the friction of the jack but assuming $\mu = 0{\cdot}3$. If a coefficient of friction of 0·35 is assumed, the calculated reduction is between 25 per cent. and 26 per cent. The test shows that the assumption of a coefficient of 0·3 is reasonable and gives slightly conservative results. The diagrams in *Figs.* 9 and 11 and the data in *Table* I are based on similar diagrams presented by Mr. Chas. E. Reynolds at the International Congress on Prestressed Concrete held in Berlin in May, 1958.

The bridge described in the foregoing was designed by the South Eastern Gas Board in collaboration with Messrs. Simon-Carves Ltd., who are the licensees in Great Britain for the BBRV system. The contractors were Sir Robert McAlpine & Sons, Ltd.

Exhibition of Precast Concrete.

An exhibition of more than 200 photographs of various types of precast concrete products was held in London in November under the auspices of the Cement & Concrete Association, and is now visiting other cities. The exhibition may be seen at the following places. Admission is free.

December 15 and 16. Houldsworth Hall, Manchester.

December 18 and 19. Bluecoat Chambers, Liverpool.

January 5 and 6. McLellan Galleries, Glasgow.

January 8 and 9. Freemason's Hall, Edinburgh.

January 12 and 13. Marryat Hall, Dundee.

January 19 and 20. Central Hall, Gosforth, Newcastle.

January 22 and 23. Corn Exchange, Leeds.

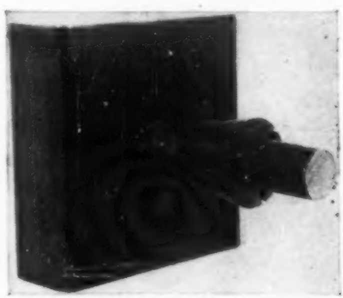
January 26 and 27. Mappin Hall, Sheffield.

January 29 and 30. Technical College, Nottingham.

February 2 and 3. Stuart & Suckling Halls, Norwich.

New Anchor for Prestressing Bars.

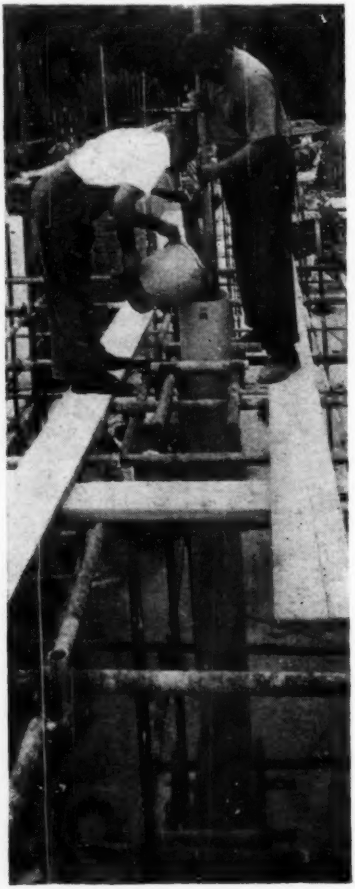
The wedge anchor-grip shown in the illustration has been developed for use with the Lee-McCall prestressing system; the wedge can be anchored anywhere on the bar, whereas a threaded end can be



used only on bars of predetermined lengths. The use of this grip enables bars of stock lengths to be used, and the end of the prestressed member is nearly flush. The recommended initial jacking stress for Macalloy bars anchored with wedge grips is 45 tons per square inch. Threaded anchors are still available.

Fibrous Material for Column Shutters.

THE illustration shows the use of a tube of fibrous material as shuttering in the construction of sixty-five reinforced concrete columns at a building at Bournemouth. When the reinforcement is fixed the tubes are lowered over it and fixed at the top and bottom and the concrete placed. Three days later a hose is used to soak the tubes with water and they are cut away. It is claimed that the cost is about two-thirds of the cost of other shutters for round columns. The architect is Mr. Alan A. Briggs and the contractors Messrs. W. E. Chivers & Sons, Ltd. The tubes were supplied by Venesta, Ltd.



Accidents during Prestressing.

THE "Annual Report of the Chief Inspector of Factories for the Year 1957" (H.M.S.O. Price 5*s.*) states that serious accidents, some fatal, have occurred when high-tensile steel has broken or has been suddenly released during tensioning in the production of prestressed concrete. If it breaks the steel whips out of the mould or concrete at a dangerous speed, and to eliminate this risk it is recommended that tunnel-shaped guards made of expanded metal or stout wire mesh should be used.

If the tensioned reinforcement is suddenly released, for example by failure of the anchoring device, it may fly out with sufficient force to impale anyone nearby. To overcome this risk, it is recommended that strongly-constructed shields of steel or timber be fitted behind each anchorage, that the jacks should be operated from the side of the beam, and that access to the area behind the anchorage should be prohibited until the tensioning operation is completed.

Strength in Bonding of Indented Prestressing Wires.

INVESTIGATIONS on the effects of repeated loading of prestressed beams showed that, when the concrete is of high strength, deformed wires offer little advantage over slightly-rusted plain wires in developing bond with the concrete and in controlling cracking. The ultimate strength of beams with post-tensioned and grouted cables, in which failure was due primarily to crushing of the concrete, may be calculated by simple methods provided that the initial tension in the cables conforms with normal practice. Associated tests on prestressing wires have confirmed that the loss of prestress resulting from relaxation of stress in the steel is likely to be between 4 and 5 tons per square inch in the case of wire from coils of large diameter, and that special treatment in manufacture can provide a wire that results in a loss of only about 1 ton per square inch. [From "Building Research, 1957." H.M. Stationery Office. Price 5*s.* 6*d.*]

Additional British Standard Loadings.

An amendment to British Standard Code No. 3, "Code of Functional Requirement of Buildings, Chapter V—Loading", issued recently contains the following additions and alterations.

Wind Pressure.—The recommendations relating to wind pressure in the 1952 edition of the Code have been extended to include chimney shafts, sheeted towers, and structures that are permanently without walls, such as braced towers, gantries, and girders. The basic wind pressure on such structures is determined by the height of the structure and the greatest probable velocity of the wind at a height of 40 ft. The basic intensities of pressure p' apply to structures of which the members have non-circular cross sections, and are given in the accompanying table. In the case of structures less than 200 ft. in height the pressures exceed those recommended for ordinary buildings of equivalent exposure; the difference is about 50 per cent. for very low structures, about 20 per cent. for structures 50 ft. high, and 10 per cent. to 15 per cent. for structures 100 ft. high. The data apply to steady wind pressures, and warning is given that certain structures, including reinforced concrete chimneys, may oscillate when subjected to the effects of the wind. Whereas the basic intensity of pressure on a building is assumed to act uniformly on the entire height, the basic pressure on a chimney or tower is assumed to vary from a maximum at the top to a minimum at the bottom; the intensity at any plane at any height depends on the height of the plane above the ground.

For structures which consist of members of circular cross section, the recommended design pressure is $0{\cdot}7p'$, but for single members of circular cross section the pressure is $0{\cdot}6p'$. In the case of isolated structures such as chimney shafts and sheeted towers the recommended design

Wind Pressure (p') on Unclad Structures, Sheeted Towers and Chimneys

Height of the part of the structure in feet above ground level	Wind pressure (p′) at any point on a structure in lb/sq. ft when the velocity in miles per hour at 40 ft above the ground is						
	50	55	60	65	70	75	80
up to 10	7	9	10	12	14	16	18
20	8	10	12	14	16	18	21
30	9	11	13	16	18	21	24
40	10	12	14	17	20	23	26
50	11	13	15	18	21	24	27
60	11	13	16	19	22	25	28
70	12	14	17	20	23	26	30
80	12	15	17	21	24	27	31
100	16	16	18	22	25	29	33
120			19	22	26	30	34
140			20	24	27	32	36
160			21	24	28	33	37
180			21	25	29	33	38
200			22	26	30	34	39
250				27	32	36	41
300				28	33	38	43
350				30	35	40	45
400				36	36	41	47
450				37	37	42	48
500 or more				38	38	43	49

(1) Structures more than 100 ft. in height should be designed for velocities not less than those to the right of the stepped line. (2) The maximum value of the mean velocity (at an effective height of 40 ft.) during a period of one minute should be used in design.

pressure is p' multiplied by the shape factor given in the Code plus 10 per cent.; that is when the ratio of height to width at the base of a shaft or tower is not less than eight the design pressure is $0{\cdot}77p'$ if of circular cross section, $1{\cdot}1p'$ if octagonal, $1{\cdot}43p'$ if of square cross section and the direction of the wind is normal to one face, or $1{\cdot}1p'$ if of square cross section and the direction of the wind is normal to a diagonal.

The smaller reduction due to shape and the greater basic pressure lead to a significant increase in design pressures compared with those for ordinary buildings. Consider, for example, a circular chimney of 5 ft. diameter, projecting 40 ft. above the roof of a building 40 ft. high, and subjected to a wind with a velocity of 72 miles per hour. Treated as a projection above the roof, the design pressure is $0{\cdot}7 \times 22 = 14{\cdot}4$ lb. per square foot; considered as a chimney shaft under the additional recommendations, the design pressures are $0{\cdot}77 \times 25{\cdot}2 = 19{\cdot}4$ lb. per square foot at the top and 16·3 lb. per square foot at roof level. The total pressure on the chimney, considered as a projection above the roof, is $5 \times 40 \times 14{\cdot}4 = 2880$ lb.; considered as a chimney shaft it is $(19{\cdot}4 + 16{\cdot}3) \times 0{\cdot}5 \times 5 \times 40 = 3570$ lb. In the first case the centre of pressure is 20 ft. above the roof and the bending moment at roof level is 57,600 ft.-lb.; in the latter case the centre of pressure is about 25 ft. above the roof and the bending moment is 73,230 ft.-lb.

Recommendations are given permitting the reduction of pressure on the leeward members of framed or lattice structures if they are partially shielded by the windward members. The notes on the permeability of buildings under the influence of wind have been elaborated.

GARAGE FLOORS.—New loadings are given for garage floors for vehicles not exceeding $2\frac{1}{2}$ tons in weight. As the loading given in the Code of 1952 (comprising a minimum imposed load of 80 lb. per square foot or an alternative minimum uniformly-distributed total load of 640 lb. per foot of width in the case of slabs and 5120 lb. per square foot in the case of beams) is not specifically superseded, it may be assumed that such floors should be designed for whichever loading gives the greatest bending moment or shearing force.

The new loadings for vehicles not exceeding $2\frac{1}{2}$ tons in weight are: (*a*) for floors used only for parking, the design load due to the most adverse arrangement of the vehicles, taking into account the actual wheel loads. (*b*) For floors used as garages other than for parking only, as (*a*) but not less than 80 lb. per square foot of floor for slabs and 50 lb. per square foot of floor for beams. This may be slightly less severe than the loading in (*a*), but it should be remembered that a floor normally used for garage services may sometimes be used for parking vehicles closely together, and it seems advisable to design all floors associated with the garaging or parking of light vehicles for the loading given in the Code or for the most adverse arrangement of vehicles.

BALCONIES AND STAIRS.—It is recommended that balconies be designed for the same loading as the floor or other area to which they give access. Although the recommendation in the 1952 Code that the minimum alternative loadings do not apply to cantilevered beams and slabs has not been specifically cancelled, the revisions imply that it is, in part at least, superseded in view of the new recommendation that the span of a cantilevered balcony shall be considered to be the projection of the cantilever when considering the alternative load. Cantilevered steps that are structurally independent should be designed to resist an alternative load of 300 lb. acting at the free end.

A Dome at Dallas, Texas, U.S.A.

In this journal for January, 1956, Mr. Charles S. Whitney described the design of a dome of 300 ft. diameter for the Dallas Memorial Auditorium (*Fig.* 1). The building has recently been completed, and the method of construction is shown in *Fig.* 2.

The dome was cast in segments, the same shuttering being used for each. A central tower, about 90 ft. high, was first

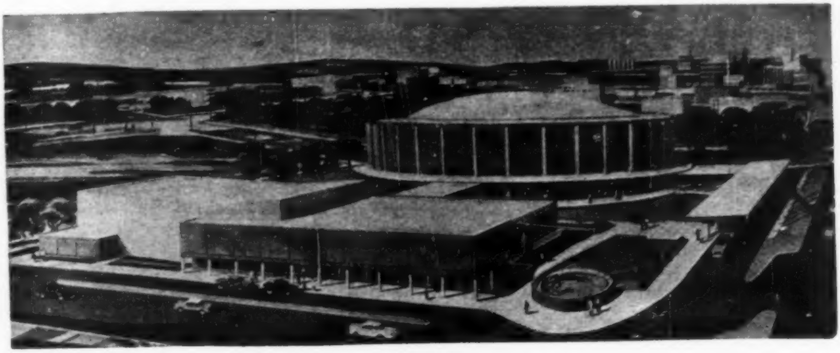


Fig. 1.

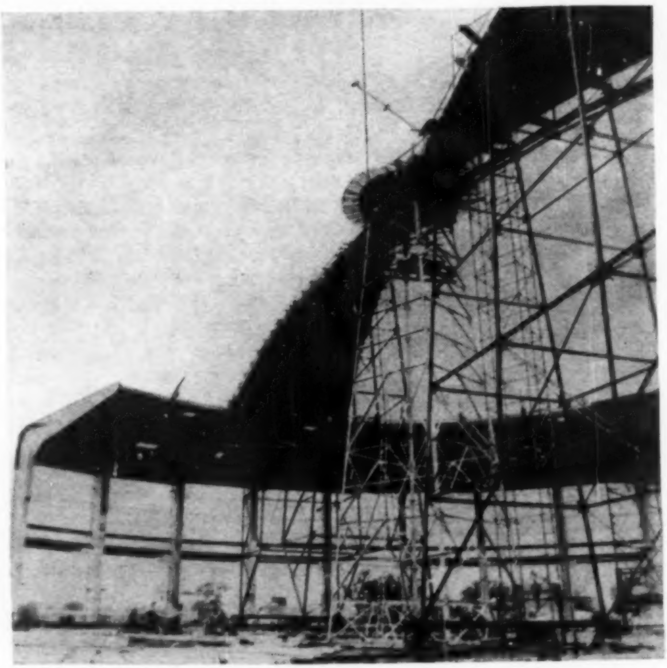


Fig. 2.—Shuttering for Opposing Sections of the Dome.

erected, on which the central part of the dome was cast. Radial bars were left projecting from this part to form a connection with the segments. The shuttering for the segments was then erected, together with a supporting steel framework mounted on rollers; each section of shuttering was supported at four intermediate points and at each end. The segments were concreted, the unbalanced thrust being resisted by the central tower until the concrete had hardened; the shuttering was then released and moved into the position to support the adjacent segments.

The architect is Mr. George L. Dahl, and the consulting engineers are Messrs. Amman & Whitney.

FIFTY YEARS AGO.

FROM "CONCRETE AND CONSTRUCTIONAL ENGINEERING", NOVEMBER-DECEMBER, 1908.*

THE illustration shows a three-story reinforced concrete transit shed, designed on the Hennebique system, in course of construction at Sandon Dock, Liverpool, in 1908. It is 628 ft. long, 78 ft. wide, and 56 ft. high, and was designed for imposed loads of 20 cwt. per square yard on the first floor, 14 cwt. on the second floor, and 3 tons on the ground floor. Electrically-operated travelling cranes of $1\frac{1}{2}$ tons capacity, with an outreach of 46 ft. from the face of the shed, were later erected on the roof. The columns are at centres of 26 ft. longitudinally and 30 ft. 10 in. and 47 ft. 2 in. transversely. On the landward side the upper floors cantilever 16 ft. beyond the ground floor. The consulting engineers were Messrs. L. G. Mouchel & Partners.

The Mersey Docks and Harbour Board inform us that, as a result of the suspension of the usual maintenance work during the war of 1939–1945, it became necessary to repair the structure; the concrete cover had spalled, exposing the reinforcement over considerable areas, and the external faces were covered with reinforced gunite 2 in. thick. The cast-iron window sashes were badly corroded, and were replaced with precast concrete sashes.

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.

New Type of Long-span Roof.

A METHOD of constructing roofs with long spans, which was developed in South America about seven years ago and has now been introduced into this country, is known as the " Silberkuhl " system, and has been used on about seventy structures in South America and Europe.

External and internal views of such a roof are shown in *Figs.* 1 and 2. It consists of conoidal slabs, which may be cast in place or precast, and tied arches of structural steel. The upper and lower booms of the arches support adjacent slabs, and are connected vertically by a lattice. The space between the booms is glazed to form a north-light. The slabs and the arches are connected structurally.

It is stated that the system is economical for clear spans of between 70 ft. and 210 ft., and that loads of up to 15 tons can be suspended from any point; other advantages claimed for the system are good and uniform natural lighting and insulation.

The roof shown in *Figs.* 1 and 2 has a span of 100 ft. and the arches are at centres of 23 ft. It supports two suspended cranes, each of 3 tons capacity, on longitudinal girders connected to the lower booms of the arches. We are informed that more than 500,000 square feet of this type of roof are now being built in Great Britain.

For spans of up to 70 ft., precast prestressed slabs are used. The units are hyperbolic (*Figs.* 3 and 4), with a thickness of 2 in. and a width of up to 8 ft. A hyperbolic surface can be generated by

Fig. 1.—Roof of 100 ft. Span.

Fig. 2.—Interior of Building shown in Fig. 1.

Fig. 3.—Prestressed Roof Slabs.

straight lines, and the prestressing wires are therefore placed along the generators (*Fig.* 4) and pre-tensioned. The slabs are placed in position and connected with concrete cast in place; the profile of a roof obtained in this way is shown in *Fig.* 5. Each slab can support a load of 2 tons.

The licensees of the system in this country are A.T.A. Industrial Co., Ltd. The steel arches are fabricated and erected by Modern Engineering (Bristol), Ltd., at whose works the buildings shown in the illustrations have been erected.

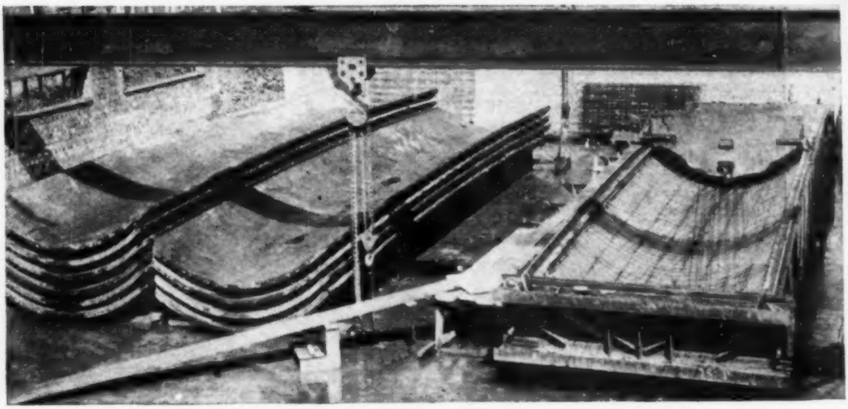


Fig. 4.—Method of Casting Slabs shown in Fig. 3.

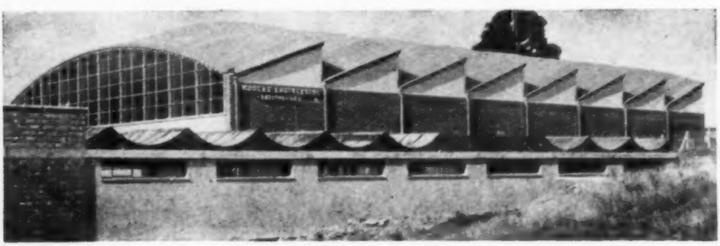
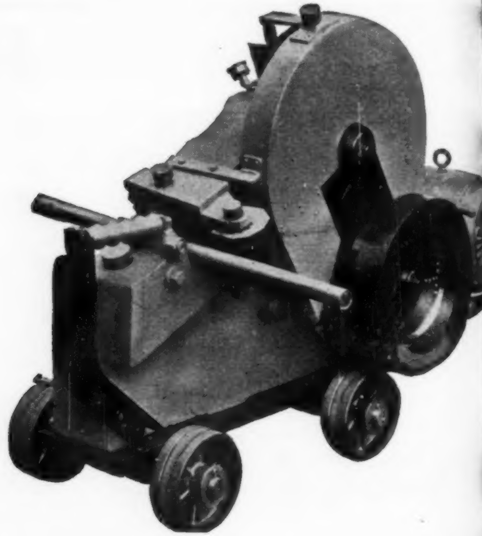


Fig. 5.—Roof shown in Figs. 3 and 4.



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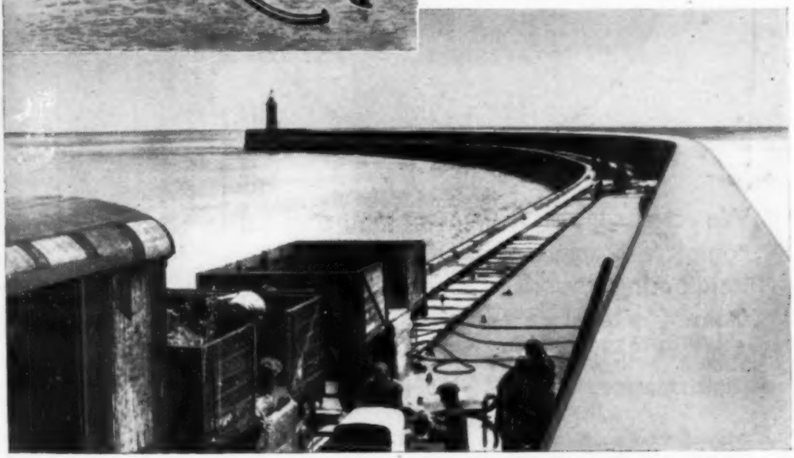

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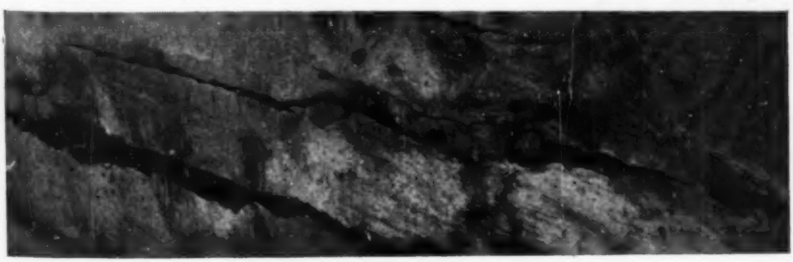

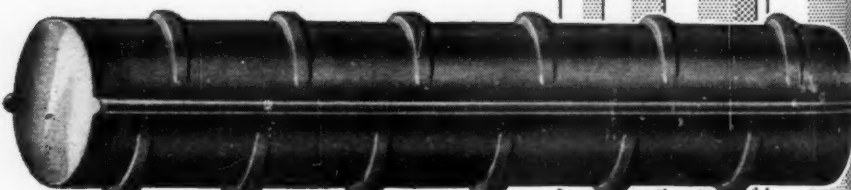

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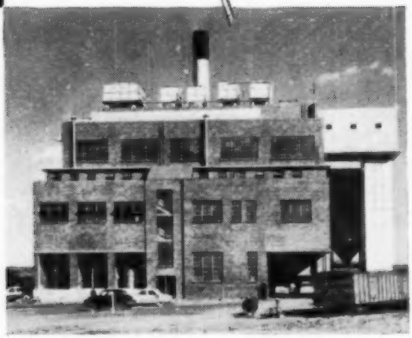

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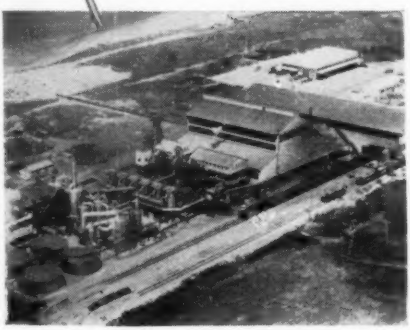

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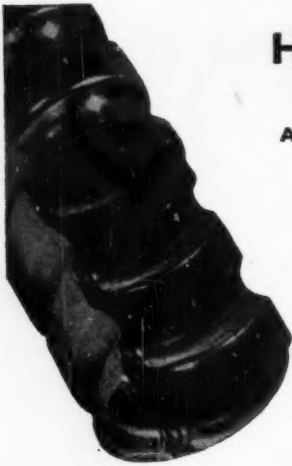

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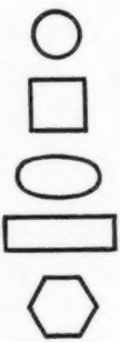

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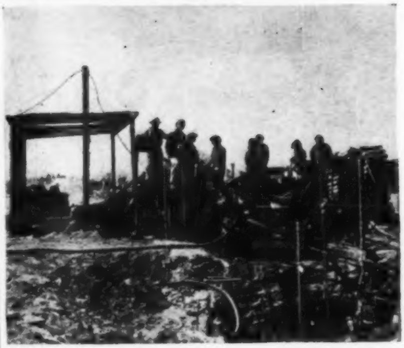
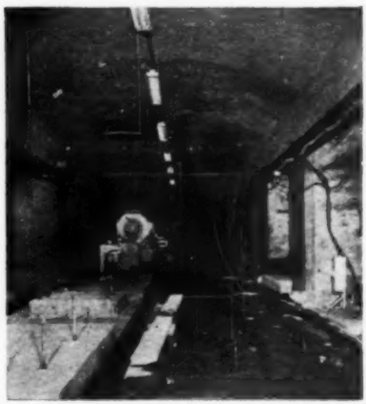


COLGROUT mixing plant at a South African Gold Mine.



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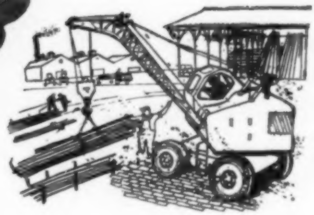

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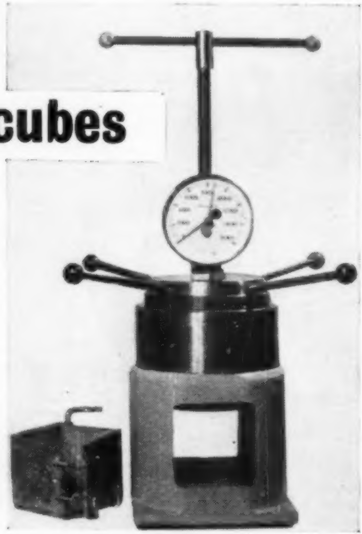

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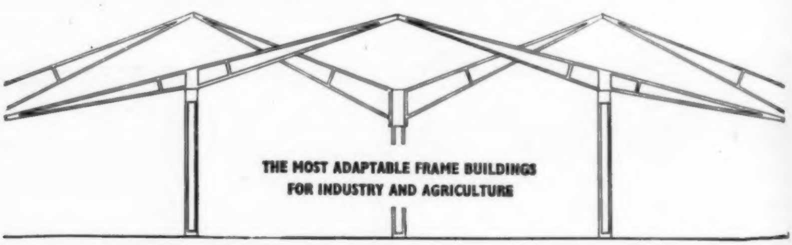

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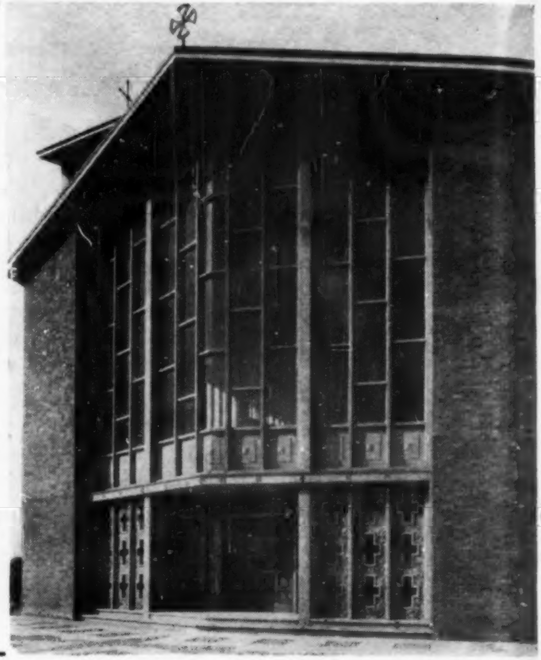

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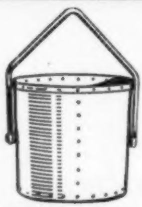

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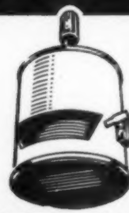

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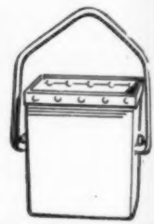

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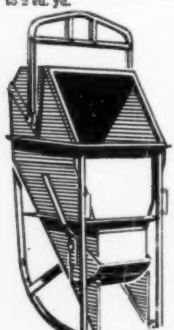

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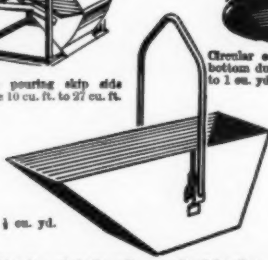

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In response to many requests, the article and tables by Mr. Jacques S. Cohen, in this journal for August, 1957, have been reprinted together with an additional table. These tables enable any rectangular section to be quickly designed by both the elastic method and the load-factor method, and the relative economy of the results of each method of design can be seen at a glance. The price of the reprint is 4*s*. (4*s*. 3*d*. by post); 1 dollar in Canada and the U.S.A.

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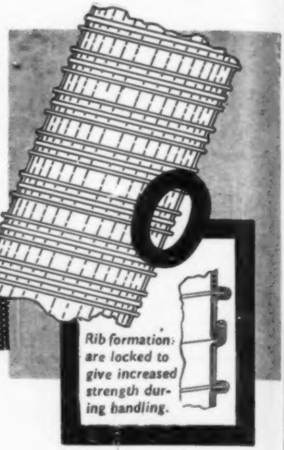

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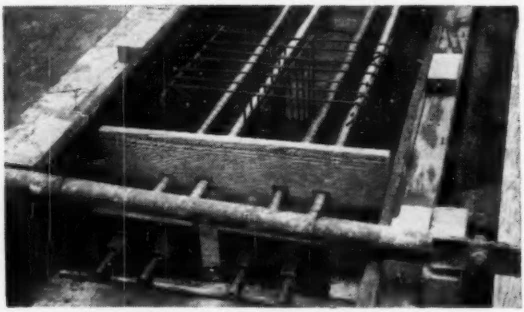

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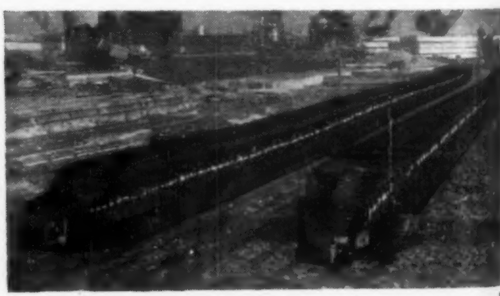

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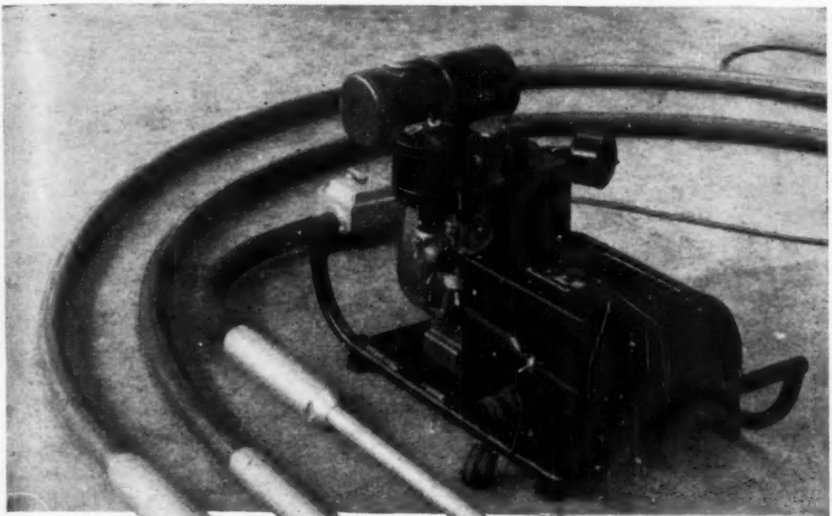

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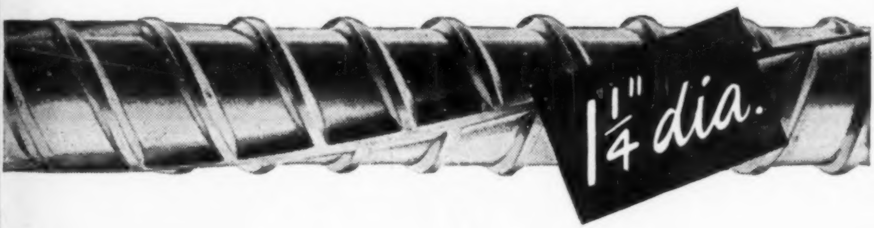

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